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FINAL REPORT:

FOR RESEARCH ON

"PERFORMANCE TESTING AND ANALYSIS OF RECYCLED AND NON-RECYCLED ADDITIVES IN ASPHALTIC PAVEMENTS: MICROSURFACING & RUBBERIZED ASPHALT"

Submitted to

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EXECUTIVE SUMMARY

The generation of large amounts of solid waste has created much concern at local and state government levels in Florida. The situation is aggravated by Florida's sensitive environment, vulnerable ground water supplies, and continuing need to find additional landfill capacity.

In 1988, the Florida legislature passed a comprehensive solid waste management bill (Senate Bill 1192), which directed the Florida Department of Transportation (FDOT) to expand, where feasible, its use of recovered waste materials in highway programs. The legislation directed FDOT to initiate research using available expertise in the Florida state university system and to build demonstration projects for determining the feasibility of using various solid wastes in construction projects. Among the waste by-products considered for road construction are ground rubber tires and crushed glass in asphalt pavements.

For these reasons, research was conducted at the University of Central Florida Circular Accelerated Test Track (UCF-CATT) by the Civil and Environmental Department involving one recycled by-product material, ground tire rubber, and one non-recycled material, microsurfacing.

A review of available literature concerning rubberized asphalt and microsurfacing was conducted. It was found that numerous states are experimenting these products trying to perfect there potential for maintenance and rehabilitation of both asphalt pavements and Portland Cement Concrete pavements.

From the review, it was also found that there are 31 facilities built in different countries for full-scale accelerated pavement testing. Of those listed, sixteen are circular test tracks, seven are linear test tracks, five are test roads for controlled vehicle loadings, and two are test pits with hydraulic loading devices. The intent of accelerated testing is to identify, understand, and characterize factors that influence pavement performance in short periods of time. These results help predict future pavement performance for design and evaluation purposes.

Chapter three describes the testing apparatus at the University of Central Florida (UCF) along with the track preparation and modifications that had to be performed in order for testing to take place. Some of the changes included the raising of the existing track slab 5" (12.7 cm) along with the bridge slabs, as well as changing steel I-beams to adjust the height of the apparatus.

Among the materials placed on the track are two mixes, one conventional S-3 and the other S-3 with 5% rubber (by weight of binder), which were placed on the test track by a PF115 paver and compacted with a steel roller. Both mixes were AC-30 binder with 7.5% asphalt content by weight. Also, two material mixtures of microsurfacing, **GRIPLAST** (without fibers) and **GRIPFIBRE** (with fibers) were delivered as a Type II mix with 100% aggregate, 12% emulsion, 0.25% to 1.5% mineral filler, and 6% water. A detailed sequence of photographs describes the placement procedure of each one of the materials being tested.

A comprehensive interwoven testing and analysis work plan was proposed. Following the literature review of previous work, the research team (professor and graduate students) began the preparation for some laboratory and experimental work. All tasks performed for completion of the research are not duplication of previous efforts by other researchers. The laboratory program investigated the performance criteria of paving mixtures. Along with the laboratory and performance tests, the analyses proposed to further evaluate the performance of each test section. Pavement analysis program such as **CHEVRON** for flexible pavement and other programs were searched for analytical work to compare with experimental test results.

Using the UCF-CATT testing apparatus, performance testing was started on all pavement sections, running only 57.25 hours over a five week period. During this time period pavement distress was monitored and documented through photographs and rut depths measurements. All pavement sections experienced polished aggregate, rutting, material uplift, and ravelling at different times and degrees. No pavements indicated cracking except for the microsurfacing's joint reflection crack located at the track slab construction joint.

From testing, the **microsurfacing** sections failed in a short period of time. Representatives for the supplier were consulted, and they offered some of the following possible reasons for the early distresses to incur:

1. Both sections had excessive amounts of emulsion and/or water in the mixes.
2. That even though no compaction is specified in the United States, in Europe compaction is required for both the tack coating and the applied pavement to squeeze out the excess water which is blocked by the concrete slab.
3. The slow-setting SS1 tack coat should have cured for a longer period of time.
4. The water that is continuously sprayed on the track to reduce tire wear tended to pond in the ruts thereby creating water pressure in the voids and flushing the fines and larger aggregate out from the binder (ravelling).
5. Due to configuration of the test track, the paver machine's wheels had to mount the tack coated concrete surface which lifts the coating off as the wheels rotated, thus removing the bond between concrete surface and the paving material.
6. For the first half of testing, impact was induced from the dual-tires contacting the timber bridge ramps which caused the tires to bounce at certain locations around the track which corresponded to microsurfacing locations.
7. Improper or unusual installation procedures used since the materials had to handworked due to the configuration of the test track.

The asphalt pavement sections favored much better than the microsurfacing, however the control sections **C1.5** and **C1** performed better than their rubberized equivalents **R1.5** and **R1**, specifically the 1.5" (3.81 cm) thick sections. Representatives for these paving mixtures were consulted, and they also offered some of these reasons for pavement distresses incurred:

1. The water that is continuously sprayed on the track to reduce tire wear tended to

pond in the ruts thereby creating water pressure in the voids (even larger for rubber mix) and flushing the fines and larger aggregate out from the binder (ravelling).

2. For the first half of testing, impact was induced from the dual-tires contacting the timber bridge ramps which caused the tires to bounce at certain locations around the track which corresponded to rubber pavement locations.

3. Due to configuration of the test track, the paver machine's wheels had to mount the tack coated concrete surface which lifts the coating off as the wheels rotated, thus removing the bond between concrete surface and the paving material.

RECOMMENDATIONS

From the results of this study, the following recommendations are given:

1. Rubberized asphalt (5% rubber by wt. of binder) should be at least 1.5" (3.81 cm) thick for all pavement system.

2. More research is needed regarding the material testing and installation procedures for the microsurfacing products to be overlayed on concrete pavements.

3. Further, more testing is required for a number of material sets both from laboratory specimens and core sampling in order to correlate the performance with optimum content of additives in paving mixture, specifications, and cost effectiveness.

CHAPTER 1

INTRODUCTION

It has been estimated that the construction, rehabilitation, and maintenance of the nation's highways annually requires 350 million tons of construction materials (natural and manufactured). This includes 20 million tons of asphalt, 10 million tons of portland cement, and 320 million tons of natural aggregates, paving mixtures, coatings, and so forth. Many state agencies have developed or are developing and implementing procedures to include a variety of waste materials in construction and rehabilitation processes [1].

The generation of large amounts of solid waste has created much concern at local and state government levels in Florida. The situation is aggravated by Florida's sensitive environment, vulnerable ground water supplies, and continuing need to find additional landfill capacity.

In 1988, the Florida legislature passed a comprehensive solid waste management bill (Senate Bill 1192), which directed the Florida Department of Transportation (FDOT) to expand, where feasible, its use of recovered waste materials in highway programs. The legislation directed FDOT to initiate research using available expertise in the Florida state university system and to build demonstration projects for determining the feasibility of using various solid wastes in construction projects. Among the waste by-products considered for road construction are ground rubber tires and crushed glass in asphalt pavements.

In regards to rehabilitation, the Federal Highway Administration (FHWA) has made efforts to promote cost-effective preventive pavement maintenance to the states and local agencies. In the past, maintenance has generally been a function of the states operations. Federal-aid funds were not eligible for maintenance activities, but the Intermodal Surface Transportation Efficiency Act (ISTEA) changed that a little. ISTEA permits states to use federal funds for preventive maintenance techniques on interstate highways when the treatment's cost-effectiveness can be proven. Now the federal personnel work with the state personnel to promote preventive maintenance of high-traffic interstate highways. It is best, in some cases, to spend a little money now on a pavement that looks like it is in good shape rather than have a big bill later to rebuild that pavement when it fails. One relatively new non-recycled material that can help in this regard is known as microsurfacing.

Accordingly, research was conducted at the University of Central Florida by the Civil and Environmental Department involving one recycled by-product material, ground tire rubber, and one non-recycled material, microsurfacing. The **objectives** of this research are to investigate:

1. The type and amount of crumb tire rubber and microsurfacing materials required in pavement mixture. Characteristics of each pavement mixture with additive will be reviewed from available literature.

2. The UCF Circular Accelerated Test Track (UCF-CATT) facility is used for performance testing of the pavement sections. The pavement sections will consist of pavement mixtures with additives and a typical pavement mixture without additive.

3. The cost effectiveness and life cycle cost.

4. The development of specifications for the pavement mixtures with additives. This specification may include the physical requirements, chemical requirements, packaging and identification requirements, and certification requirements. The specification will not address any safety or environmental concerns associated with this study.

It should be noted that ground tire rubber, crushed waste glass, and fibrous asphalt were the materials originally proposed, but both crushed waste glass and fibrous asphalt mixtures were dropped from consideration. Crushed waste glass was dropped based on information from a literature review and local inquiries made on the cost factors incurred, such as availability, proximity, hauling, crushing, plant retrofitting and anti-stripping agent. Fibrous asphalt was dropped mainly because the paving company expressed concern in being able to properly mix the required polypropylene fibers in the heated asphalt with their existing plant setup. For those reasons, it was decided to make ground tire rubber and microsurfacing as the research materials of focus. A literature review concerning microsurfacing was completed on January, 1996.

CHAPTER 2

LITERATURE REVIEW

MICROSURFACING

In 1994, **Raza** [2] reported that micro-surfacing was a mixture composed of polymer-modified asphalt emulsion (quick-setting type), 100% crushed mineral aggregate, mineral filler, water, and field control additive as needed. Mineral filler was generally Type 1 portland cement, but most non air-entrained cement types could be used. Hydrated lime was also used in a few systems. Field control additive was used to adjust the break time during the field application.

Microsurfacing is basically a type of slurry seal with a polymer-modified binder and often higher quality aggregates. Although slurry seals can be placed only 1.5 times as thick as the largest size aggregate in the mix (due to high asphalt content), microsurfacing can be placed in relatively thick layers due to the increased stability of the mixture. Compared to hot-mix asphalt (HMA), which is workable when hot and hardens upon cooling, microsurfacing is mixed and applied at ambient temperatures using emulsions. The emulsion breaks and hardens through an electro-chemical process and by the loss of water from the system. Microsurfacing is also called a cold mixed system.

The most common uses of microsurfacing are surface texturing/sealing and rut filling on asphalt concrete pavements. Some States have used microsurfacing for other purposes as well. These include:

- Correcting raveling/flushing.
- Leveling course.
- Interlayer.
- Crack sealing/filling.
- Void filling.
- Pothole patching (small and shallow type).

Although microsurfacing is primarily used on asphalt pavements, some States have used it on Portland Cement Concrete (PCC) pavements and bridge decks for the restoration of skid-resistant characteristics. At least one State has used microsurfacing for filling ruts on PCC pavements.

Microsurfacing was first developed in Europe, generically known as micro asphalt concrete. In the mid 1970's, Screg Route, a French company, designed a micro asphalt concrete that was subsequently improved by the German firm Raschig. Raschig marketed its product in the United States under the trade name "Ralumac" during the early 1980's.

Microsurfacing was first introduced in the state of Kansas in 1980. Subsequently, major user states include Kansas, Ohio, Oklahoma, Pennsylvania, Tennessee, Texas, and Virginia. It has also been applied on several turnpikes in New Jersey and Pennsylvania and other freeways in various other states.

The major differences among the various microsurfacing systems are due to the types of emulsifiers and polymers used. Although microsurfacing can be designed with either anionic or cationic types, all of the emulsion used to date for microsurfacing in the United States have been cationic. Some of the systems are commonly known by the trade name of the emulsions, for example: Ralumac, Polymac, Macroseal, and Durapave.

Since microsurfacing, like other thin surface treatments, is intended for functional improvements, no structural design is performed. The microsurfacing design process consists of the following steps:

1. Selection and testing of mixture components to verify whether they meet the materials specification.

2. Mixture testing to determine (a) mixing and application characteristics of the two major constituents (i.e., emulsion and aggregate), effects of water content, and effects of filler and additives and (b) optimum asphalt cement content.

3. Performance related tests on mixture samples to ensure good long-term performance.

Aggregates (excluding mineral filler) constitute about 82% to 90% by weight of the microsurfacing, depending on the aggregate gradation and application, and have a strong influence on the microsurfacing performance. For best results, the aggregates should be 100% crushed, clean, strong, and durable particles free of absorbed chemicals, clays, and other materials that could affect bonding, mixing, and placement. The crushed aggregate should be angular and not have too many flat or elongated particles. Aggregate gradations and other mixture components normally follow International Slurry Surfacing Association (ISSA) recommendations.

Mineral filler serves two major purposes: (a) to minimize aggregate segregation and (b) to speed up or slow down the rate at which the system breaks and sets. For most aggregates, mineral filler shortens the break time. Portland cement and hydrated lime have been used as mineral filler for microsurfacing. Mineral filler typically increases the stiffness of the asphalt residue. For most aggregates, mineral filler is required for the mixture to set properly. Mineral filler, particularly portland cement, may also be used to improve gradation, but the cost may be prohibitive. Normally up to 3% of portland cement (or 0.25% to 0.75% of hydrated lime) by dry weight of aggregate is specified.

Polymer-modified cationic asphalt emulsions are currently used for microsurfacing mixtures in the United States. The residual asphalt content of microsurfacing generally varies from 5.5% to 9.5% of the dry weight of aggregate. Properties of an asphalt emulsion greatly depend on the chemical known as the emulsifier. The emulsifier keeps the asphalt droplets in stable suspension and permits breaking at the proper time. As the amount of emulsifier is increased, the break time is generally increased.

Water is the mixing medium for the microsurfacing mixtures. It is the main factor determining mixture consistency. It is introduced in three ways: as moisture already in the aggregate, as mixing water, and as one of the two major constituents of the emulsion. All potable water can generally be used for microsurfacing. Depending on the weather condition and aggregate absorption rate, good microsurfacing mixtures can be placed over a limited range of total moisture content, typically 4% to 12% of the weight of the dry aggregate. Lower amounts of mixing water are used during cold weather, and higher amounts are used during hot weather. Mixtures containing more than 12% water may become too fluid and segregate as evidenced by the settling of the aggregate and floating of the asphalt.

The addition of polymers typically increases the stiffness of the asphalt and improves its temperature susceptibility. Increased stiffness improves the rutting resistance of the mixture in hot climates and allows the use of a relatively softer base asphalt cement, which in turn provides better low temperature performance. Polymer-modified binders also show improved adhesion and cohesion properties. An amount of 3% to 4% polymer solids (present in distillation residue) by the weight of asphalt residue is typically specified for microsurfacing mixtures. Generally, an increase in polymer amount (up to a limit) will increase the mixture stiffness. Some laboratory studies indicate that the addition of polymers will usually result in maximum stiffness at an asphalt emulsion content of about 10% to 12%. The polymers used in microsurfacing are the same as used for other asphalt mixes. Natural rubber latex is used most often, but other polymers, including styrene-butadiene rubber (SBR), styrene-butadiene-styrene (SBS), and ethylene-vinyl-acetate (EVA), have also been used. The amount and suitability of polymers is currently determined by viscosity and softening point tests on the asphalt cements.

An additive may be used to either accelerate or retard the break time of microsurfacing mixtures. Generally, the emulsifier used in emulsion manufacturing is used as an additive because of its compatibility with other mixture components. The amount of additive ranges from 0% to 2% by the volume of emulsion. The common practice is to keep the quantity of additives low.

Microsurfacing should not be placed if either the pavement or air temperature is below 10°C, if it is raining, or if there is a forecast of ambient temperature below 0°C within 24 hours of placement. Some projects have failed when placed in cold and/or wet conditions. If placed in cold weather, microsurfacing may ravel and crack. If placed in very hot, dry weather, the surface can break too fast, causing water retention and slowing interior curing. Hot weather requires a formulation change for longer mixing times to enable the microsurfacing to be properly applied.

For high volume roads, a self-propelled, front feed, continuous loading and mixing machine is used to place microsurfacing. These machines are capable of receiving materials from nurse trucks while continuing to mix and apply the mixture. The mixers for microsurfacing machines are to allow a thorough blending of materials into a homogeneous mixture. Excessive mixing time may result in stripping of the asphalt from the aggregate. Mineral filler is added to the aggregate just before it enters the mixer. Water and additives are combined and added to the aggregate as it falls into the mixer. These ingredients are mixed before the emulsion is introduced, usually at about the one-third point of the mixer.

All pavement joints and cracks that are 6 mm or wider should be repaired and sealed before the application of microsurfacing. To ensure proper curing of repair work, all surface cracks, joints, and potholes should be repaired 1 to 6 months before microsurfacing is applied. It is desirable to keep crack sealant below or flush with the surface. A tack coat is not required unless the pavement surface is extremely dry, raveled, or made of concrete. ISSA recommends the tack coat should consist of one part asphalt emulsion and three parts water and should be applied at a rate of 0.16 L/m^2 to 0.32 L/m^2.

During hot weather, the pavement is usually prewetted to control a premature breaking of the emulsion and to improve bonding with the existing surface. Much of the success of the construction of microsurfacing depends on the knowledge and skill of the crew that operated the machine as a traveling cold mix plant. If the mix is too stiff, it may prematurely set in the spreader box or will drag under the strike-off. If it is too fluid, the mixture may segregate or run into channels, and binder-rich fines can migrate to the surface, resulting in uneven surface friction. Slightly drier mixtures generally perform better than wetter mixtures.

Microsurfacing surface courses are usually applied in thicknesses of 10 mm to 15 mm. The basic goal is usually to place the material with a thickness that is at least 1.25 times the nominal size of the aggregate in the mixture. Microsurfacing is designed so that the system can sustain rolling traffic after one hour of application.

The use of microsurfacing on PCC pavements is not widespread. However, it has been used to improve skid resistance of PCC pavements and bridge decks. Microsurfacing applied directly on the PCC pavement may start to debond within a relatively short period. To ensure a proper bond with an existing PCC surface, it is required that a tack coat precede the application of microsurfacing.

Microsurfacing costs vary depending on many factors including location, availability of quality materials and contractor, application rates, maintenance of traffic, and other bid items. Microsurfacing is approximately two to three times the cost of hot mix asphalt concrete on a weight basis, $75/ton to $135/ton. Since its unit cost is higher, the cost-effectiveness of microsurfacing is dependent on the concept that thinner applications can be utilized. Thinner applications also reduce adjustments to curbs, shoulders, drainage inlets, bridge expansion dams, and guardrail. When used for filling wheel ruts, microsurfacing cost-effectiveness depends on negating the need for usually used combined milling and overlay operations.

States generally believe that microsurfacing is a prudent and cost-effective technique for texturing and filling wheel ruts on high volume roads.

Jones, [3] in 1995 reported that Federal Highway Administration (FHWA) officials recently invited the International Slurry Surfacing Association to present a microsurfacing demonstration in McLean, Va. The test strips were two 500-foot long sections of asphalt drive that were treated with microsurfacing, a thin surface treatment of polymer-modified slurry seal.

The demonstration at the FHWA research center was part of the agency's newly emphasized efforts to promote cost-effective preventive pavement maintenance to the states and local agencies. The material was laid on a section of the research center's circular asphalt driveway that was built in the 1970's and overlaid in about 1982. The underlying pavement was not that bad but it did have some minor transverse and longitudinal cracking.

Slurry Pavers, Inc., laid the microsurfacing with the supervision of a vice president of ISSA. The job used a continuous-run, front-feed Bergkamp Mobil-mix Paver fed by a Flow-Boy support truck with a capacity of 20 plus tons of aggregate and 600 gallon each of emulsion and water. The mix design called for a normal CQS-1H natural-latex-modified Ralumac from Asphalt Emulsion, Inc., and Type I portland cement. The target residual asphalt on the Type III emulsion was 6.7%; on the Type II emulsion the residual asphalt content was 7.4%. For the demonstration, a single lift of Type III at approximately 22 lb/yd^2 (117 N/m^2) using the standard microsurfacing overlay box with a secondary strikeoff used to take out any deformities that might come out of the regular spreader box. The Type II material was a finer gradation of aggregate. An 18 lb/yd^2 (95.8 N/m^2) was put down with the same spreader box used with the Type III, but without the secondary strikeoff.

It was a horrible day, weather-wise. It was completely overcast, and there were thundershowers in the general vicinity. The standing water was swept off the pavement and then put down the microsurfacing. The microsurfacing "broke off" in just a few minutes and had a good hard set within an hour, it could have been opened to traffic. A severe thunderstorm came through a couple of hours later, hail and everything else, and there was no apparent damage to the material. The county road advisers seemed enthusiastic about seeing the microsurfacing demonstration. Several of the county road advisers at the annual FHWA meeting had never actually seen it and knew little of its application techniques.

Hixon, [4] in 1993 reported that a technique to level a rutted pavement and provide surface friction was microsurfacing. In 1983 as part of a FHWA demonstration project, the Oklahoma Department of Transportation (ODOT) contracted its first microsurfacing project. Since then, the use of microsurfacing has increased each year. As with a slurry seal, no rolling of the microsurfacing was required for either rut filling or surfacing. Standard practice for ODOT consisted of applying microsurfacing directly to the surface of a structurally sound asphalt concrete pavement in one or two passes with a laydown machine.

Three projects were established to provide an in-depth study of microsurfacing at ODOT. These were monitored for at least 3 years following microsurfacing. The typical section for all three projects was very similar with a 4.5" (11.4 cm) of asphalt concrete on 8" (20.3 cm) to 12" (30.5 cm) of fine aggregate bituminous base. Several other projects were examined to monitor special uses.

These include microsurfacing over fabric, over portland cement concrete, and using synthetic latex.

Microsurfacing was also used as an interlayer to fill cracks and level the pavement before the application of a paving fabric and a 6" (15.2 cm) asphalt concrete overlay at ODOT. Six years after the completion, the project was in excellent condition, with no reflective cracking.

ODOT had limited experience with microsurfacing on PCC pavements. ODOT applied microsurfacing to PCC bridge decks but did not modify the surface before any treatment. After 4 years, there was significant raveling in the wheelpaths. On another project, the existing PCC pavement was treated with a tack coat of emulsion prior to microsurfacing. This patch showed limited raveling after 3 years of service.

In 1987 a project was performed as an experimental evaluation of a new form of microsurfacing emulsion. The new emulsion used a synthetic latex and different emulsifier and set retardant than did the traditional microsurfacing mix. This project showed that 100% reflective cracking after 3 years but improved the friction values after 3 years of service.

Typical costs of microsurfacing on projects in Oklahoma from 1983 to 1991 ranged between \$77/ton and \$109/ton. Average application rates ranged from 21 lb/yd^2 (111.7 N/m^2) to 37 lb/yd^2 (196.8 N/m^2) for rut filling, on the basis of a treatment 6' (1.8 m) wide for the outside lanes only. Average application rates for surface treatments have from 23 lb/yd^2 (122.4 N/m^2) to 32 lb/yd^2 (170.2 N/m^2), on the basis of a treatment 12' (3.7 m) wide. Costs of microsurfacing with those of cold milling and a 1.5" (3.8 cm) asphalt overlay showed microsurfacing to be only 55% of the cost of the asphalt overlay. However, estimated lives for the two treatments were 5 years for microsurfacing and 10 years for the asphalt overlay. On the basis of annual costs and the estimated life cycles, the asphalt overlay becomes slightly more effective.

Conclusions on the use of microsurfacing from ODOT were:

- Reduces the level of rutting and retards the rate of rutting, compared with pretreated rutting levels, after 4 years of service.
- Provides good friction characteristics of the pavement for up to 9 years of service.

- Has shown a moderate resistance to reflective cracking.
- Does not increase the load-supporting ability of a pavement.
- Can be used to fill ruts up to 1.5" (3.8 cm) deep.
- Works well for filling depression cracks and alligator cracks.
- Has generally not been successful when placed as a surface course over fabric.
- Works well as an interlayer.
- Has worked successfully with mine chat (cherty limestone) and a dolomite-granite aggregate mixture.
- Has shown limited success on PCC.

Microsurfacing is recommended for:

- Continued use as a maintenance tool on both Interstate and state highway asphalt pavements.
- Filling ruts and reestablishing the transverse profile of an asphalt roadway.
- Restoring pavement friction characteristics.
- Filling wide depression and alligator cracks.

Kazmierowski (1993), [5] from Ontario, Canada, reported the following advantages when using microsurfacing:

- Less energy is expended because the microsurfacing is applied at ambient temperatures; in addition, there are also no harmful emissions often associated with hot-mix production.

- Because the thickness of a lift of microsurfacing is typically 9 mm to 12 mm versus 40 mm for a hot-mix overlay, the result is a significant conservation of nonrenewable resources, asphalt cement and aggregates.

- The thin surfacing does not significantly alter the road profile; therefore, the need for guide rail adjustments, the reduction of bridge clearances, and the need to reinstate shoulder granulars are greatly reduced.

- The cost is approximately 60% to 65% of a single-lift hot-mix overlay.

- Compared with surface treatment, there is not a problem with loose aggregates damaging vehicles.

The disadvantages of microsurfacing were:

- It does not inhibit reflective cracking or provide structural support.
- Placement must be during warm, dry weather (at least 10°C).
- An experienced contractor and proper mix design are critical to the success of the process.

Three contractors participated in the demonstration project. Contractor A, Elsamex system from Spain; Contractor B, Ralumac system from Germany; Contractor C, Micromat System produced in the United States. Before construction, the roadway had slight frequent longitudinal cracking and coarse aggregate loss. The pavement exhibited extensive moderate transverse cracking, intermittent alligator pavement edge cracking, and intermittent slight to moderate centerline cracking.

Contractor A used a material somewhat gap-graded; Contractor B used a coarser material and it was slightly gap-graded; Contractor C used the most open-graded material. The type of polymer modifier used by Contractor B and Contractor C was natural rubber, while Contractor A use a synthetic latex. A certain amount of modifier was required to achieve the desirable temperature susceptibility of the binder.

On the basis of the short-term results, the following specific conclusions were made:

- Microsurfacing provides a practicable alternative to a one-lift overlay on roadways with surficial deficiencies and a structurally sound base in a freeze-thaw environment; it provides a uniform, dense-graded, highly skid resistant surface. Microsurfacing exhibits potential to address performance deficiencies in skid resistance, raveling, and coarse aggregate loss.

- High-quality construction practices (skilled crew and specialized equipment) coupled with a comprehensive mix design process are crucial to ensure the success of microsurfacing.

- All three contractors designed and placed an acceptable microsurfacing product on this project. All three products have exhibited similar performance results to date.

- The high-quality surface course mix aggregates on the ministry's designated sources list can be used in a microsurfacing mix.

- Microsurfacing does not eliminate reflection of structural cracks.

- The single-load method with only one truck-mounted mixer is too slow to be used on large jobs, particularly on heavily trafficked highways.

Kuennen, [6] in 1992 reported that new high-tech, proprietary surface treatments are on the verge of entering the U.S. market from France. Although these techniques offered superior wearing resistance and almost astonishing durability, they came at a higher price.

Briefly, very thin (20 mm to 25 mm) and ultra-thin (10 mm to 15 mm) wearing courses of surface materials were being used in France, as the result of government pavement construction and maintenance policies. These policies provided high road service levels, and involved the addition of polymer, mineral or fiber modifiers.

Jean Lefebvre is one of the largest road contractors in France, including a U.S. office right now. Its group, **GRIPFIBRE**, is a cold applied, proprietary thin wearing course incorporating synthetic fibers. The mixture is mainly used for maintenance. Homogenous, pourable across a large width, it contains flexible, synthetic fibers. Ten million to fifteen million fibers per square meter offer strong resistance to all kinds of attacks, and includes **GRIPFIBRE's** exceptional durability. Manufacturing and placement are made with a machine equipped with a continuous mixing unit, towering an extendable spreader box, articulated at its center and of adjustable height.

RECYCLED CRUM RUBBER TIRE ASPHALT

Fager, [7] in 1994 reported on eight rubber hot bituminous mix projects from 1990 to 1992, constructed on the Kansas Department of Transportation highway system. Four were dry process and other four were of the wet process. Approximately 616 tons of rubber was used in the hot mix overlays. The projects allowed Kansas DOT to experiment with mix variations. Standard virgin mixes, new gap-graded mixes, and even recycle mixes were tried on the projects. The primary method of determining the amount of asphalt and rubber had been the Marshall method.

Preliminary conclusions from the Kansas DOT projects were as follows:

- From the crack survey results, it is apparent that the rubber may not inhibit the development of cracks in the higher-density mixes. However, even though the results are still preliminary, the gap-graded mixes show the greatest potential in reducing the amount of cracking.

- Rubber in a gap-graded mix will prevent the asphalt draining off the aggregate during construction. This will allow a thicker film thickness on the aggregates

- None of the rubber projects have rutted, but neither have the asphalt-only control sections.

- On hot recycle projects, rubber addition rates should be based on the weight of the dry virgin aggregate and recycled asphalt pavement.

- Rubber absorbs a large portion of an RA-100 in a hot recycle mix. An AC-5 binder with rubber will reduce the asphalt absorption and improve the aggregate coating.

Hanson (1994), [8] reported that in 1991 Mississippi placed a conventional HMA (control) along with a rubber-modified HMA (RMHMA) test section. Crumb rubber was incorporated into the pavement using a recent wet process technology known as the continuous process developed by Rouse Rubber Industries. This process blended the fine powdered crumb rubber, which pass a No. 80 sieve screen and had a No. 200 sieve mean particle size, with asphalt cement.

Laboratory test results indicated that the RMHMA mix had lower water susceptibility and higher tensile strength and resilient modulus. The gyratory properties and creep/permanent deformation tests indicate that the RMHMA mix should be more resistant to rutting. In-place performance was evaluated by visual observation of surface cracking and rut measurement for the first 2 years of the pavement's life. There was no cracking in either the control or the RMHMA test section. The amount of rutting was insignificant and was likely caused by densification of the mixes.

When crumb rubber material (CRM) is reacted with asphalt cement, a thick, elastic, viscous, and adhesive material called rubber-modified binder is formed. The rubber-modified binder produced by the traditional McDonald process also has a higher softening point and lower temperature susceptibility. The swelling of the CRM increases the adhesive property of the rubber-modified binder.

In general, an RMHMA has a lower Marshall stability and a higher Marshall flow than a conventional HMA. The Marshall stability of RMHMA mix has been shown to decrease by as much as 60% and the Marshall flow of RMHMA mix has been shown to increase by as much as 4.2 times the control mix.

On the basis of laboratory tests, the fatigue resistance of RMHMA is better than conventional HMA. Adding 5% CRM to HMA will probably increase the fatigue resistance of a pavement to twice that of a conventional asphalt mix. The researchers found that the fatigue performance of RMHMA mixes was improved at higher rubber-modified binder contents. At temperatures greater then 60°F (15.6°C), all RMHMA mixes performed better than the control mix.

The resilient modulus of RMHMA mixes is about 75% of the control mixes. The resilient modulus of RMHMA mixes is lower than that of conventional mixes but only at low temperatures, below 75°F (23.9°C). At temperatures higher than 75°F (23.9°C), the resilient modulus of RMHMA mixes are higher than that of conventional mixes. RMHMA mixes with fine CRM have higher resilient modulus values than mixes modified with coarse CRM. RMHMA mixes with higher CRM content have lower resilient modulus values. At a low CRM content, the fine CRM may have a more significant effect on the resilient modulus of the mixes.

The RMHMA mixes investigated have lower creep resistance than conventional asphalt mixes. This difference is more pronounced at higher test temperatures. It has also been found that mixes with fine CRM gradation have better creep resistance than mixes with coarse CRM gradation. However, test results from dynamic creep testing indicate that RMHMA mixes have higher resistance to permanent deformation than conventional HMA mixes. These studies showed that under constant load the RMHMA mix deforms more than the control mix, whereas under repeated load the RMHMA mix deforms less than the control mix.

Ruth (1992), [9] reported that in 1988, the Florida Legislature passed Senate Bill 1192 which set forth in Section 336.044 of the Florida Statutes a directive to the Florida Department of Transportation (FDOT) to expand, where feasible, its use of recovered (waste) materials for highway construction. Specifically, the bill directed that an investigation be conducted to determine how ground tire rubber (GTR) from waste tires could be used in quality asphalt concrete mixtures for highway construction by undertaking demonstration projects as part of the currently scheduled construction program.

A demonstration project constructed on SR 60, Hillsborough County, was used to evaluate the performance of asphalt-rubber in these applications. As a result of this demonstration project, the FDOT has permitted the use of GTR in selected surface treatment and interlayer construction. Also, the FDOT currently permits the use of GTR in certain joint sealers and in railroad crossing pads.

In 1989 and through 1990, three demonstration projects were constructed to evaluate the use of GTR in asphalt concrete friction course mixtures. The first demonstration project used a dense-graded friction course (FC-4) containing 80 and 40 Mesh, 3.1%, 5.3%, and 11.1% GTR (by weight of asphalt cement) and binder content of 7% to 8.2%. Dense-graded friction course mixtures are generally more susceptible to changes in binder content and particle size of GTR than open-graded mixtures. Tests indicated that the mix with 80 Mesh 5.3% GTR and 7.3% binder content appeared to be the best mix. Although all of the asphalt-rubber mixtures exhibited some degree of sticking to the paver's screed, it was only considered excessive during paving of the section which had 11.1% GTR. Otherwise, no major problems were encountered during construction of these asphalt-rubber friction courses.

The second demonstration project used 80 and 24 Mesh, 5.3% to 20.5% GTR (by weight of asphalt cement) and 6.3% to 11.4% binder content in an open-graded friction course FC-2. Results obtained from construction of this demonstration project indicated that 10% to 15% GTR can effectively be used in open-graded friction course mixtures, but the total binder content should probably be less than those used in the mixtures of this project. The third and last demonstration project using 80 Mesh 10% GTR (by weight of asphalt cement) and 7.78% binder content was constructed without any major technical problems.

The constructability and short term performance of these asphalt pavements indicate that it is feasible to use GTR for friction course construction without any major change in construction operations. There is sufficient test data and corroborating information that suggest asphalt-rubber friction courses, particularly open-graded, have improved durability over conventional friction course mixtures. Developmental specifications were included in the report to serve as a guide for any additional construction projects.

Consequently, the specifications, as presented in that report, were believed to be conservative with respect to time-temperature aspects of blending and storage. A report on emissions including hot-mix operations was submitted by the FDOT to the Florida Department of Environmental Regulation (DER) Bureau of Air Regulation for their review and action. Their response stated that the Bureau had no objections to the FDOT's use of Asphalt-Rubber Friction Courses. Since the term "Asphalt-Rubber" as defined in D8-91 by ASTM stipulates a minimum GTR content of 15%, the FDOT has changed to the term "Rubber Modified Asphalt Binder." Therefore, the FDOT specifications use this revised terminology.

In 1992, **Krutz [10]** conducted a laboratory research program to assess the potential benefits of rubberized asphalt concrete mixtures. The aggregates used in the research program were a 100% crushed granite that has no history of stripping problems with in-service pavements. Three grades of unmodified neat asphalt were used: AC-5, AC-20, and AC-40. Both the AC-5 and AC-20 were then modified with crumb rubber. The AC-5 was also modified with rubber and an extender oil, yielding a very soft third modified binder. The rubber used in this research program was ambient ground rubber having a hydrocarbon content of approximately 45% and a specific gravity between 1.10 and 1.20. The resulting modified binders were:

- AC-5 + 17% Rubber (AC-5R).
- AC-5 + 16% Rubber + 5% Extender Oil (AC-5RE).
- AC-20 + 16% Rubber (AC-20R).

Optimum binder contents for both unmodified mixtures AC-5 and AC-20 were agreed upon at 5.3% and 5.7% by total weight of mix, respectively. Binder content used in preparing modified mixtures and sample preparation (% by weight of total mix) were:

- AC-5R at 8.5%.
- AC-5RE at 8.3%.
- AC-20R at 7.9%.

The first test used was a modified version of the proposed ASTM creep test. The second test used to assess permanent deformation was a triaxial, repeated-loading confined test. The testing conditions used were static load at 77°F (25°C), static load at 104°F (40°C), repeated load at 77°F (25°C), and repeated load at 104°F (40°C). From analysis of static permanent deformation testing, the rubberized mixtures show decreasing strain with increasing binder viscosity . It can be concluded from this testing procedure conducted at 77°F (25°C), the addition of rubber yields mixtures that exhibit less deformation. From the static testing at 104°F (40°C), the rubberized binders indicated the same response for a mixture incorporating rubber. All rubberized mixtures exhibited less strain than the AC-20. It is hypothesized that the rubber is absorbing the load and the strain is therefore independent of the base asphalt cement. All three of the rubberized mixtures showed a smaller drop in stiffness than the AC-20. This would indicate that rubberized mixtures will suffer a smaller loss of stiffness with increasing temperature than will unmodified mixtures.

The following conclusions can be drawn from this project:

- The addition of ground tire rubber to asphalt concrete mixtures results in mixtures that exhibit less permanent deformation at high temperatures compared with unmodified mixtures, remembering that the rubberized mixtures contained higher than optimum asphalt contents. This proved to be true for both static and repeated load testing.

- Permanent deformation testing should be carried out at elevated temperatures. This conclusion is supported by both the static and repeated load test results. The relative ranking of strain changes for both testing conditions when the 77°F (25°C) test results are compared with the 104°F (40°C) test results.

- Permanent deformation testing should incorporate repeated loading. This is not only a better model for including the effects of moving wheel loads, but is supported by comparing the static testing at 104°F (40°C) to the repeated load testing at 104°F (40°C). The static test results indicate only the presence of rubber and nothing about the properties of the base binder. The repeated load testing indicates, in a concrete manner, the differences that exist between binders.

Maupin, [11] in 1992 reported that rubber might be used in asphalt concrete as an aggregate, or it might be reacted with the asphalt cement to yield asphalt rubber. Most of the rubber that was used was asphalt rubber. The success of this product was somewhat mixed: some agencies have expressed overwhelming satisfaction with it, and others expressed doubt about the economic advantage of adding rubber to asphalt. The use of rubber has changed since its inception; therefore, there is still a need to experiment using the recent changes to determine how it forms.

An experimental field project using asphalt rubber concrete was installed in Fairfax County, Virginia, in an urban area in which slow-moving traffic often causes excessive permanent deformation. The lane sections carry 30,000 to 40,000 vehicles/day (5% to 15% trucks). A subcontractor was hired to supply the crumb rubber and to blend it with asphalt cement and an extender oil at the hot-mix plant concurrently with the hot mix operation. The AC-30 asphalt cement containing extender oil was heated to approximately 420°F (215.6°C) before being mixed with the crumb rubber, and the resultant binder was stored at 360°F (182.2°C) before being mixed with the aggregate in the hot mix drum plant.

The asphalt rubber mixtures failed all of the tests. These test results indicate that the asphalt rubber mixtures may deform if the traffic is severe enough. Because the properties of the binders may be similar at high temperatures but very different at lower summer pavement temperatures, performance of the asphalt rubber mixtures may have been predicted better by performing the traffic compaction simulation at lower temperatures. It was expected that the addition of rubber would increase the modulus and decrease the unrecovered strain; however, the asphalt rubber mixtures had a lower modulus and a higher unrecovered strain compared with the same properties of the control mixtures.

There appeared to be an optimum rubber content at which the maximum value of resilient modulus and indirect tensile strength was achieved for the fine rubber mixture for both resilient modulus and indirect tensile strength. This optimum condition was not apparent for the coarse rubber mixture. The magnitude of the resilient modulus for the coarse rubber appeared to increase as the rubber content was increased. This apparent trend could not be confirmed.

ACCELERATED TESTING OF PAVEMENTS

The intent of accelerated testing is to identify, understand, and characterize factors that influence pavement performance in short periods of time. These results help predict future pavement performance for design and evaluation purposes. There are several research facilities set up for pavement research. An Organization for Economic Cooperation and Development (OECD) report (1985) **[12]** lists 31 facilities built in different countries for full-scale pavement testing. Of those listed, sixteen are circular test tracks, seven are linear test tracks, five are test roads for controlled vehicle loadings, and two are test pits with hydraulic loading devices.

Most of the circular test tracks have the same basic configuration, consisting of an apparatus with several arms that have wheels at the ends of the arms. The diameters of the tracks range from 12 feet (3.7 m) to 131 feet (39.9 m). Top wheel speeds are anywhere between 12 mph (19.3 km/h) and 62 mph (99.8 km/h), with a maximum frequency of load repetition of 5200 per hour. Wheel loads are induced either by gravity or with hydraulic devices. The loading mechanisms usually have load through half an axle, either in single or tandem configuration. Maximum loads for the different configurations are: 22,500 pounds (100 kN) dual wheel, 31,000 pounds (137.9 kN) tandem wheel, and 33,700 pounds (149.9 kN) full axle. The wheels travel around a circular path, guided by the apparatus. They are driven either electrically or hydraulically, with motors at either the wheels or at the center of the devices.

Among the circular test tracks, two are located in the United States. One built between 1963-1964 is at Washington State University in Pullman, Washington. However, this facility is no longer in operation. The other, which is not listed in the OECD report, is at the University of Illinois at Urbana-Champaign (Barenburg, 1976) **[13]** and is a small indoor facility which was completed in 1963. The two largest tracks, built in Poland and Czechoslovakia, are 131 feet (39.9 m) and 105 feet (32 m) in diameter and have top speeds of 44 mph (70.8 km/h) and 40 mph (64.4 km/h), respectively. They are two-armed structures which apply loads through a full axle.

Two other large tracks are located in France and Switzerland. The French machine, driven by a large motor at its center, has an outer track 115 feet (35 m) in diameter as well as a secondary inner track, 57 feet (17.4 m) in diameter. It has four arms which are aerodynamically shaped. The Swiss facility has three arms and is driven by electric motors at each wheel.

The many smaller test tracks, 40 feet (12.2 m) diameter or less, are housed indoors. These are located in Canada, Finland, and Mexico. These testing facilities provide climate control and some can regulate the ground water table level. The Canadian apparatus can produce artificial rain and freeze-thaw cycles. Other known circular test tracks are located in New Zealand, Brazil (new), Germany (inactive), Soviet Union (Leningrad and Kiev), Rumania, and The Netherlands.

Several linear test tracks are also listed. Most of them are small enough to be housed indoors. The ones that are built indoors have the capability for climate and temperature control. An advantage with linear testing is that straight line motion more effectively simulates traffic on the highway, since the turning motion is not present as with the circular track. Maximum wheel load capability is the same. However, one disadvantage is that the rapidity of loading is less than what can be achieved in a circular track. This is due to the constant acceleration and deceleration needed for reversing direction. Maximum frequencies of loading range from 300 to 1800 per hour. Other linear tracks are located in Switzerland, Denmark, Germany, and two in England.

A portable machine, called Accelerated Loading Facility, is located in Sydney, Australia. This machine utilizes gravity to stop and reverse the directions at the ends of its travel. It accomplishes this by having the loading cart travel up an incline to stop and reverse its direction. The design greatly reduces power consumption. This equipment has been in use by the Federal Highway Administration in McLean, Virginia.

Another kind of mobile machine, the Heavy Vehicle Simulator (HVS), was built in South Africa toward the end of the 1960's. The HVS consists of an extra heavy vehicle that uses a beam to load a dual wheel over a 26 foot (7.9 m) length at up to 14 mph (22.5 km/h). Wheel loads can reach

22,500 pounds (100 kN) for pavement or 45,000 pounds (200.2 kN) for airport runways. Four of these machines were built, and the three newest ones are currently in use.

Other test tracks consist of special roads that have trucks and control vehicles riding over them. This is similar to the AASHO Road Test. One of these is located at Pennsylvania State University. It has an oval-shaped track one mile (1.6 km) in length. Two quarter-mile (.4 km) long test sections are incorporated in the track.

The Public Works Research Institute in Tsukuba City, Japan uses five trucks which are remote-controlled. This facility applies a very realistic loading, but only in the range of 1000 to 1500 per day. Similar facilities in Italy and Finland are set up to measure the response of pavement under various types of vehicles. It relies on several types of strain gauges in the different layers of the pavement to monitor the action.

Other testing uses machines which apply vertical loads, often with hydraulic rams. The application of loads is not through a rolling wheel, and thus less realistic than the other kinds of testing. Theses types of tests are defined as laboratory testing. Most of the state DOTs and many university laboratories have these machines setup.

CHAPTER 3

TESTING APPARATUS AND TRACK PREPARATION

UCF-CATT

The UCF Circular Accelerated Test Track (UCF-CATT), see Photograph 3.1 and Figure 1 and , funded by FDOT to test bridge expansion joints in 1989 is 50 foot (15.2 m) in diameter to centerline of 4 foot (1.2 m) width concrete slab pavement that was made to accommodate a dual wheel assembly consisting of half an axle. An average of 15 inches (38.1 cm) of pavement thickness was designed based on AASHTO Design Guide to carry 288 million equivalent 18 kips (80.1 kN) axle loads. Two proposed bridge sections (denoted as near and far) with 12 foot (3.7 m) span length were built, diametrically opposed, into the circular track.

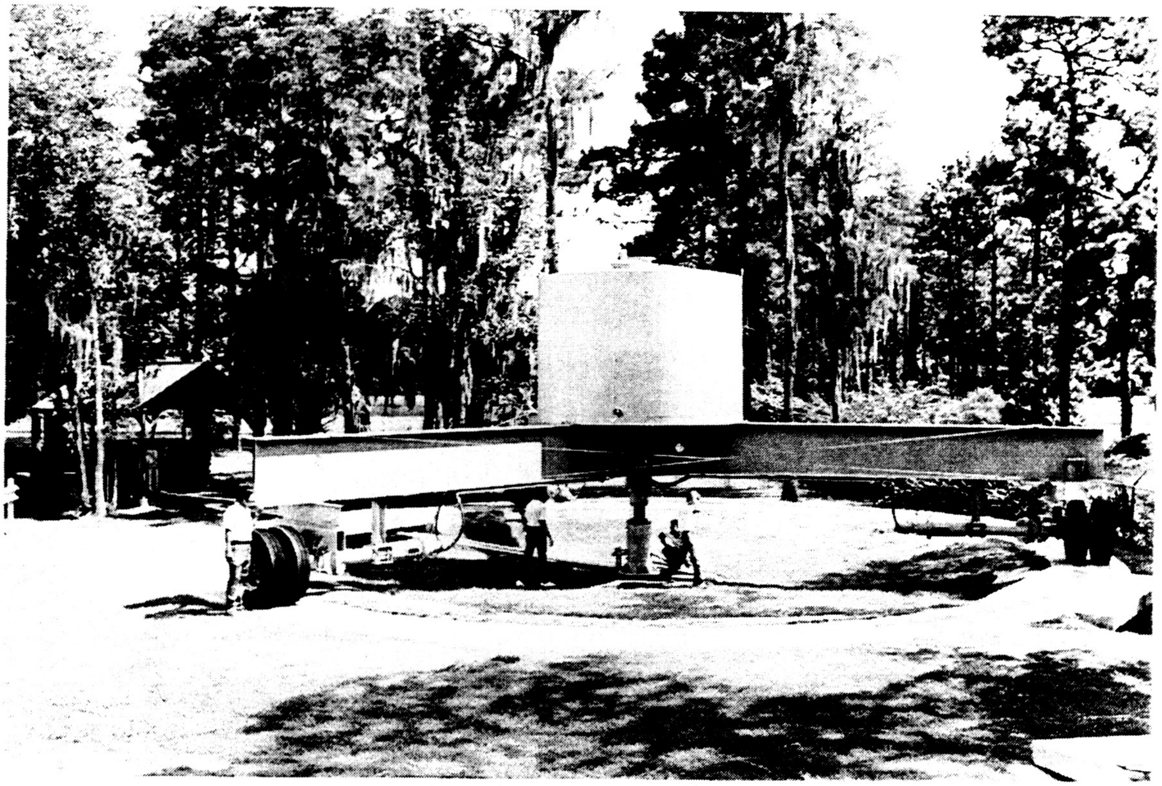
Concurrent research involving a second agency made the UCF-CATT existing track concrete slab surface too low to accept the paving sections and maintain a flat riding surface during testing. To raise the riding surface to the required 5" (12.7 cm), various depths of new concrete are laid on top of the existing track slab , and new apparatus structural steel was placed, and both the near and far bridges were fitted with timber ramps.

The operation of this test track is to have three sets of dual truck wheels travelling around a circular path, guided by three radial arms. These radial arms are 25 foot (7.6 m) W36 x 150 I-beams anchored to a center support pivot at 120 degree intervals. This specially designed central support and bearing assembly at the center of the track holds the entire system in place, while allowing the machine to float up and down, and a small angle of tilting during the course of operation, but provides no loading support except its own weight. Three axles from a tractor's tandem set of drive axles were assembled and attached to three supporting I-beams. Each axle was strengthened and modified to be driven hydraulically. A 60-horsepower (44.7 kW) hydraulic motor is mounted to each axle to deliver the power through a planetary gear speed reducer and handle dual-wheel loading up to 30,000 pound (133.5 kN).

A 7500 gallon (28.49 m^3) water tank, 12 foot (3.7 m) diameter by 8 foot (2.44 m) high, is centrally mounted on top of the support I-beams. Depending on the water level in the tank, the total load of the structural weight and

filled water tank can vary between 30,000 pounds (133.5 kN) and 80,000 (355.9 kN) pounds that will be evenly distributed to the three dual-wheel assemblies.

Power supplied by a hydraulic pump is driven by a 220 horsepower (164.1 kW) diesel engine. Hoses to the hollow central support carry hydraulic fluid to a swivel joint mounted on the top of the central support. These hoses branch out to feed each of the three driven motors. This hydraulic transmission provides the speed of rotation machine up to 30 miles per hour (43.3 km/h) in either clockwise or counter-clockwise direction. The entire facility has been used to test many large-scale bridge expansion joints since 1990.



Photograph 3.1: UCF Circular Accelerated Test Track (UCF-CATT)

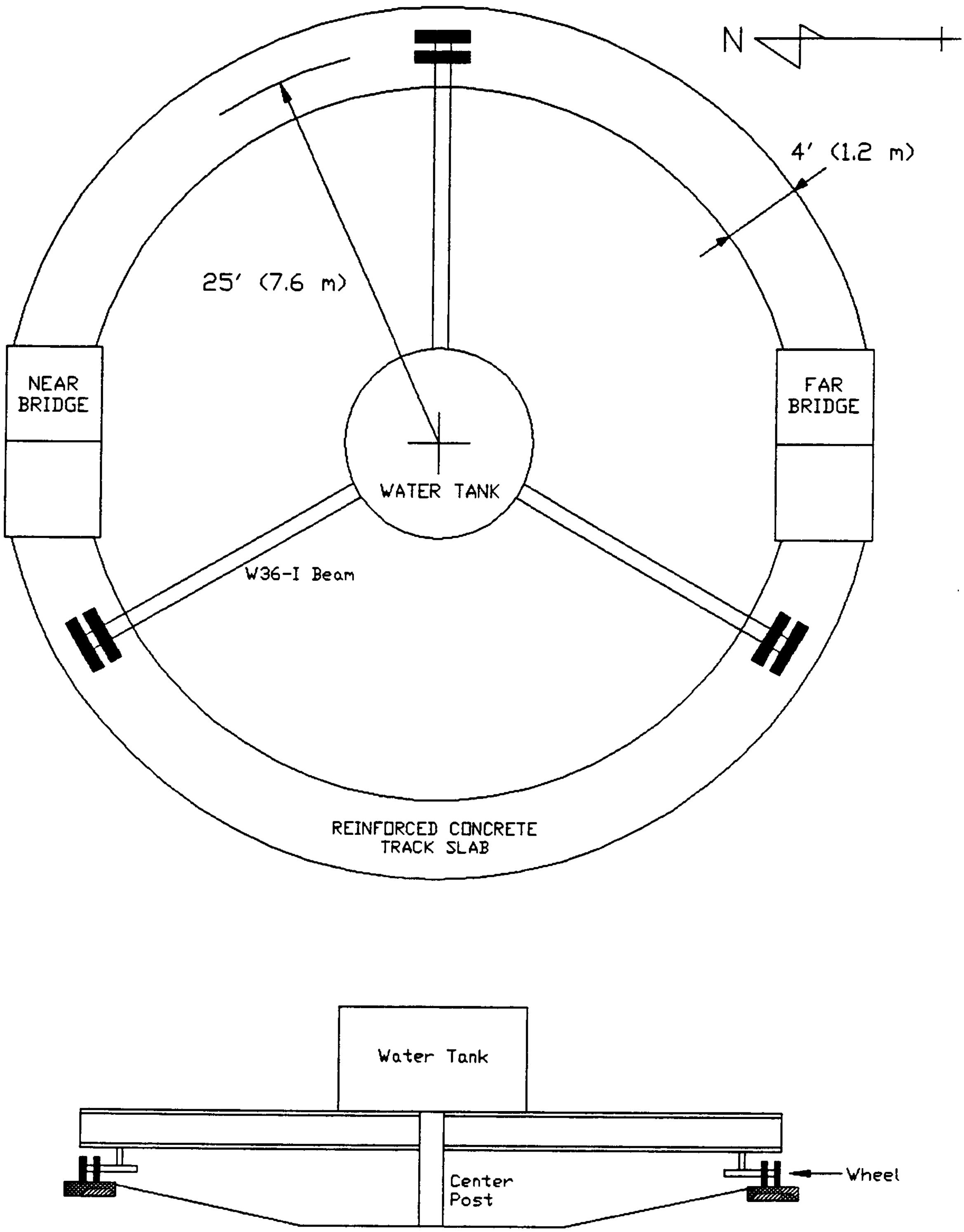


Figure 1 : Sketches of the UCF-CATT.

NEW TRACK SLAB

In order to accommodate for other projects at the same time, it was necessary to raise the existing track slab an additional 5 inches (12.7 cm) in height. The west half of the track was prepared for new concrete pouring on by excavating and forming around both sides of the existing track slab. The slab was cleaned with water and then brushed. A chemical wash followed using muriatic acid, which was later cleaned off with water. Locations of the existing through cracks within the reinforced concrete track slab were measured and shown in Figure 2, indicating potential reflection cracking in the corresponding test sections.

The following day, bonding agent (HIBOND, a polyvinyl acetate polymer emulsion) was applied and cured for 20 minutes. FIBERMESH MD fibers were added to a ready-mix concrete truck containing 6.5 yd^3 (5 m^3)of mix at the site (by the bag at 1.5 lb/yd^3 (0.9 kg/m^3) or 0.1% by volume). This particular blend contained 25 individual fiber designs with aspect ratios ranging from 40 to 85.

This new concrete was cured for seven days using sprayed water and a plastic cover. The outdoor temperatures ranged from 25 °F (-3.9 °C) to 65 °F (18.3 °C) during this cure process. The cured concrete is shown in Photograph 3.2.

Six concrete cylinder specimens, four with fibers and two without fibers, were collected while the concrete was poured. The cylinders are prepared for compression strength testing according to ASTM C 39. Two slump tests (ASTM C 143) were also performed on site for the fiber concrete only (see Table 1).

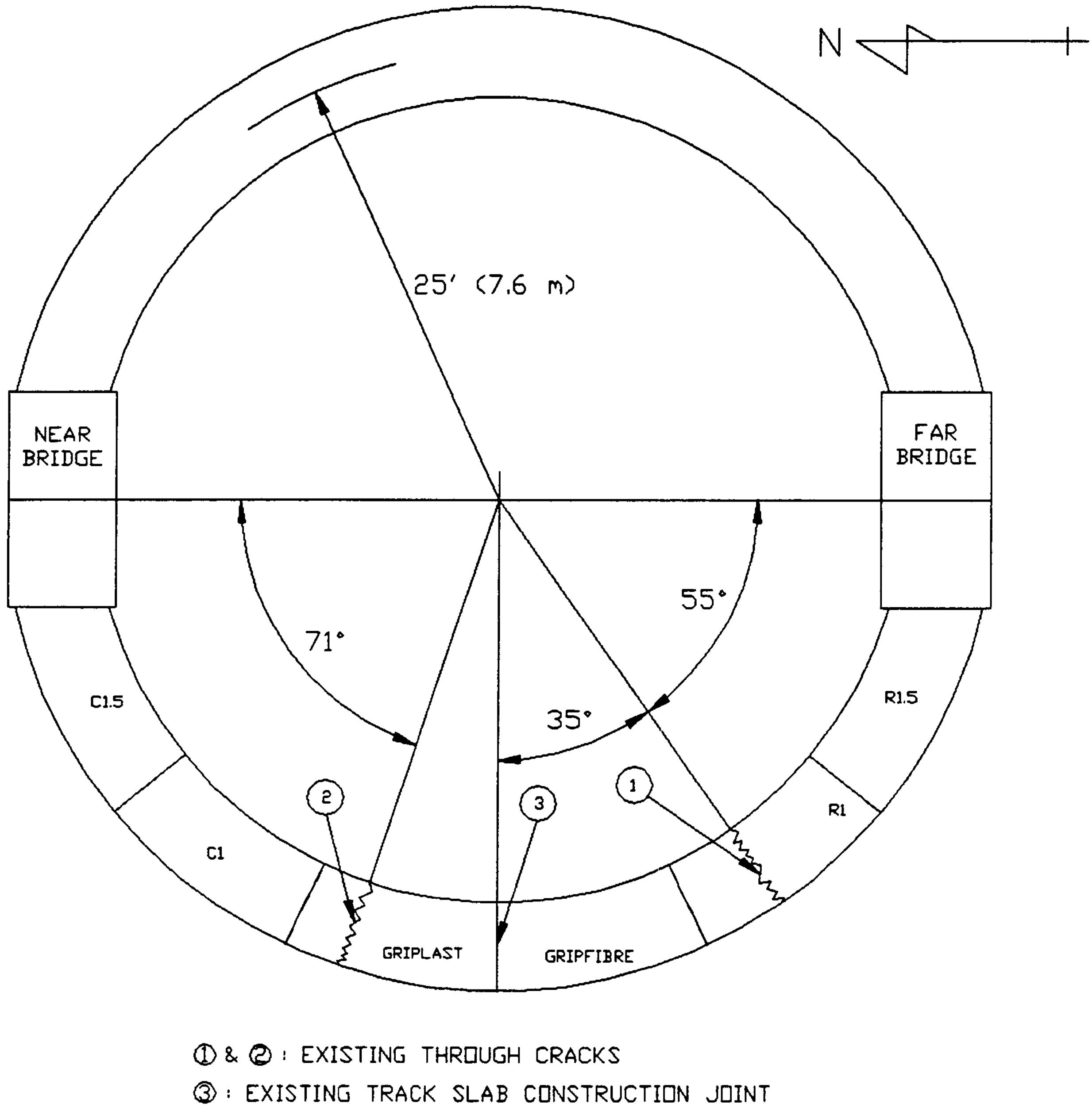
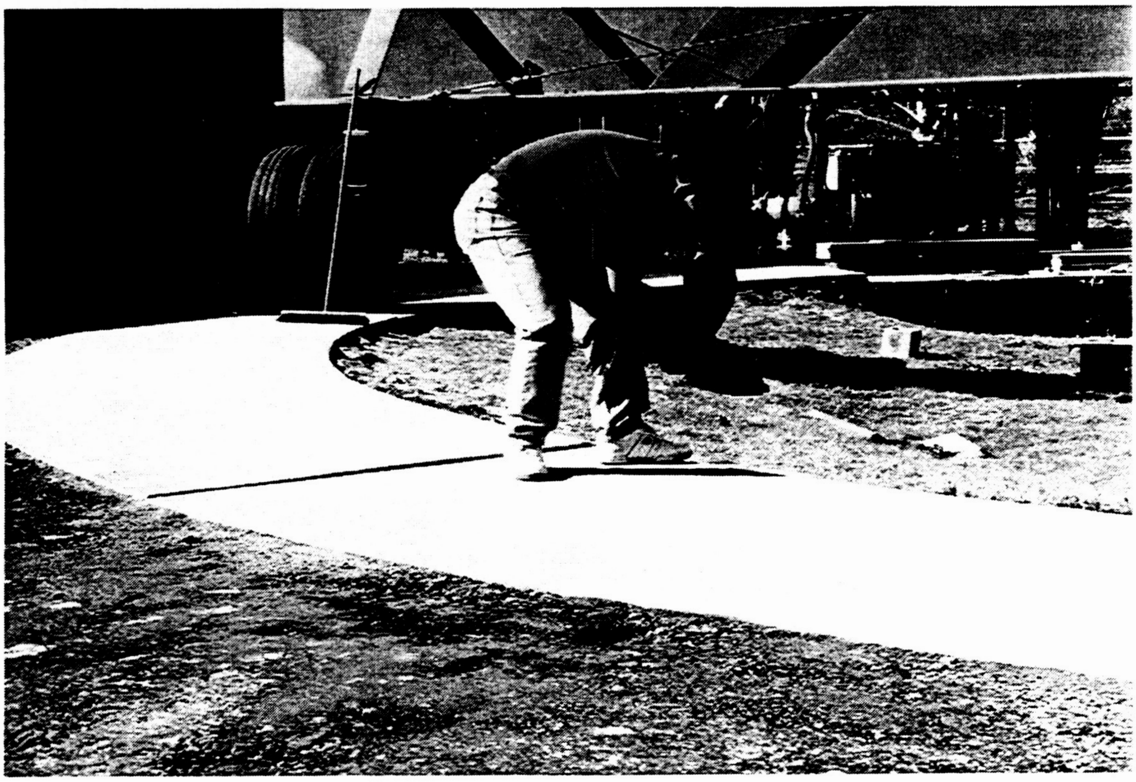


Figure 2 : Location of Existing Through Cracks in Track Slab



Photograph 3.2: Completed new fiber concrete track slab.

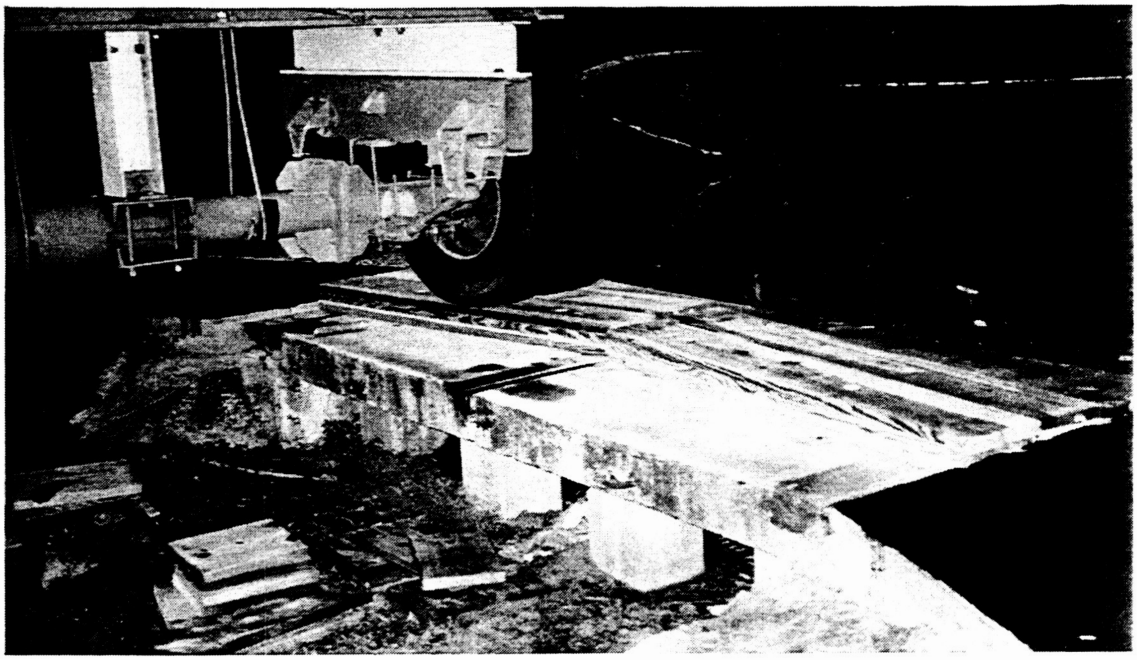
TABLE 1: NEW FIBER CONCRETE SLAB

COMPRESSIVE STRENGTH (ksi) (3" X 6" Cylinders)		
Day	With Fibers	Without Fibers
7	5.36	6.53
14	6.08	****
28	6.51	6.44
	6.10	****
SLUMP TEST (inches)		
	With Fibers	Without Fibers
Trial 1	4.50	****
Trial 2	4.75	****

1 inch = 2.54 cm; 1 ksi = 6.895 MPa

TESTING APPARATUS

After the 5 inches (12.7 cm) of the track surface had been raised as mentioned in the previous section, it was necessary to modify the six steel I-beams above the axles in order to adjust the original machine design. For that same reason, the original bridge elevation also required a 5 inch (12.7 cm) raise. It was proposed to use ramps to support the dual-wheels. Five inch (12.7 cm) high ramps, shown in Photograph 3.3, were made up of 2"x12" (5.1 cm x 30.5 cm) pressure-treated timber, laid flat, and anchored to the existing slab. However, the wood ramps did not work as planned and created a noticeable impact to the track each time the wheel entered and exited the ramps, with sensible vertical vibration in central post foundation. In order not to damage the testing apparatus, testing was suspended on for two weeks until a solution is found. The solution was a new 5 inch (12.7 cm) concrete slab poured right over the existing bridge slab having several threaded bars embedded in the existing slab anchored to the new poured slab. Since then, the impact and vibration problems were resolved.



Photograph 3.3: Timber bridge ramps.

CHAPTER 4

MATERIAL PLACEMENT

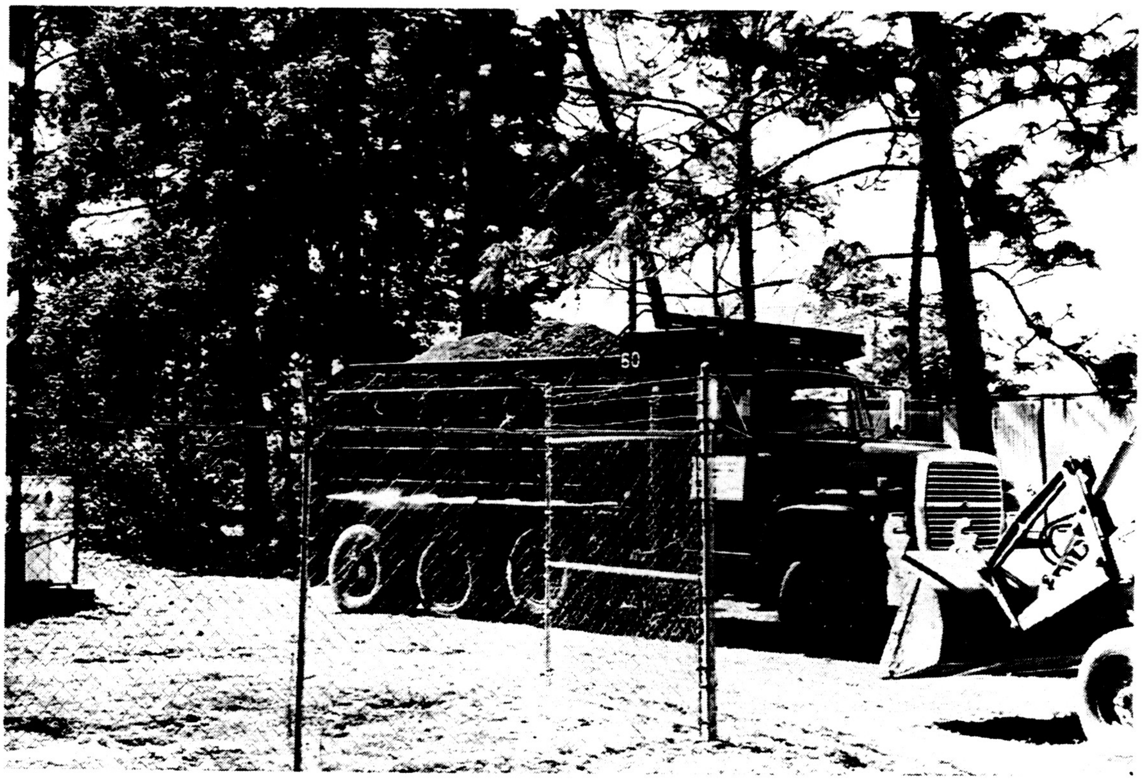
Before laying the testing asphaltic material, a local paving company was invited to prepare the west half of the test track by building up and widening the track slab to fit a regular paver machine. This was accomplished with three 25 ton truck loads of recycled asphalt pavement (RAP) and placed by a CASE 580K backloader, as shown in Photographs 4.1 and 4.2. The RAP was compacted with a 3 ton BOMAG roller, as shown in Photograph 4.3. An SS1 tack coat was applied to the concrete surfaces (see Figure 3 and Photograph 4.4) at pavement sections on **C1.5**, **C1**, **R1** and **R1.5** before the asphalt mixtures were placed. This tack coat is a very light application of a slow-setting anionic (electro-negatively charged asphalt globules) emulsified asphalt (asphalt cement and water) for a bond between the concrete slab surface and the overlaying pavement course.

CONVENTIONAL AND RUBBERIZED ASPHALT CONCRETE MIXES

Two mixes, one conventional S-3 and the other S-3 with 5% rubber (by weight of binder), were placed on the test track by a PF115 paver and compacted with a steel roller (see Photograph 4.5). Both mixes were AC-30 binder with 7.5% asphalt content by weight. Table 2 and Table 3 present the asphalt mix data.

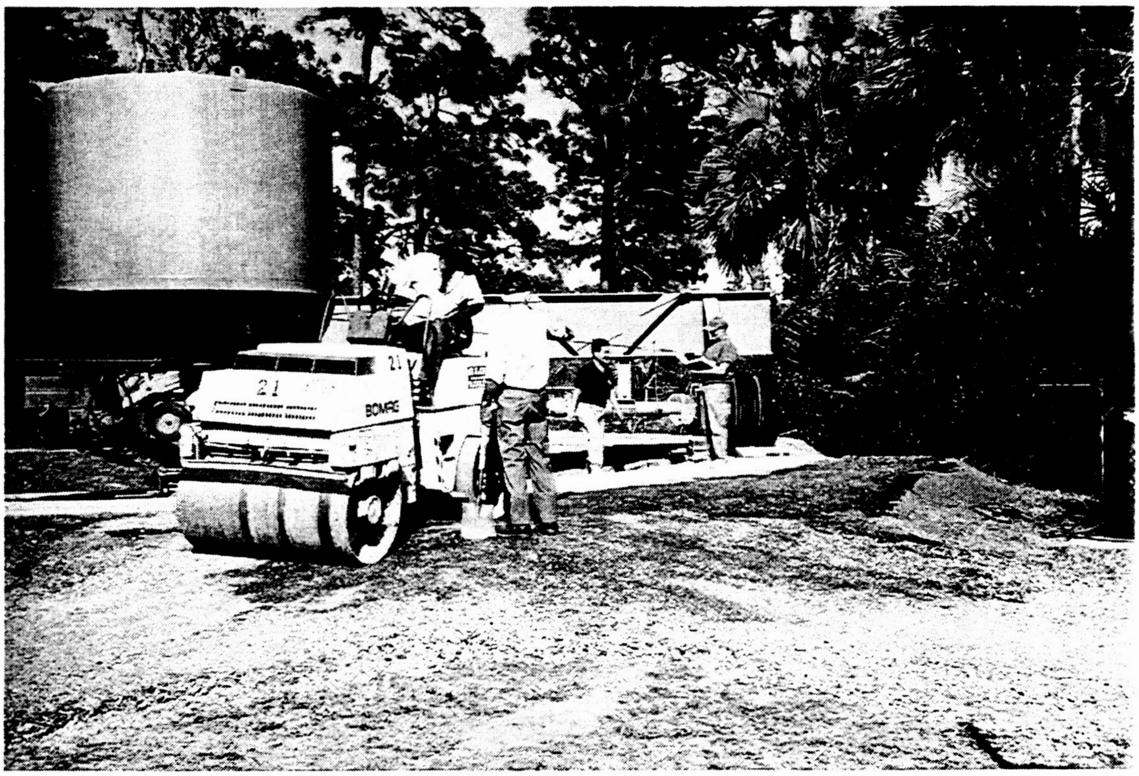
Conventional asphalt mixture was placed on sections **C1.5** and **C1 by** hand, worked with rakes, and then compacted to the required depths of 1.5" (3.8 cm) and 1" (2.5 cm) respectively. The sequential procedure were shown in Photographs 4.6, 4.7, and 4.8 respectively. A portable vibrator was used at the edge of section C1.5, where the roller drum could not reach for a full pass. The compacted asphalt density was checked using a 3411-B Troxler Nuclear Density Gage. The average density measured was 134.5 pcf (21.1 kN/m^3) which is 98.46% of normal specimen density.

Rubberized asphalt mixture was placed on sections **R1.5** an **R1** using the same procedure as used for the conventional mixture, as shown in Photographs 4.9, 4.10 and 4.11. The average asphalt density was read 131 pcf (20.6 kN/m^3). Figures 4 and 5 show the pavement sections and cross-sectional thickness.



Photograph 4.1: Truck load of recycled asphalt pavement (RAP).

Photograph 4.2: Case 580K Backloader placing RAP around the track slab.



Photograph 4.3: BOMAG Roller compacting the RAP.

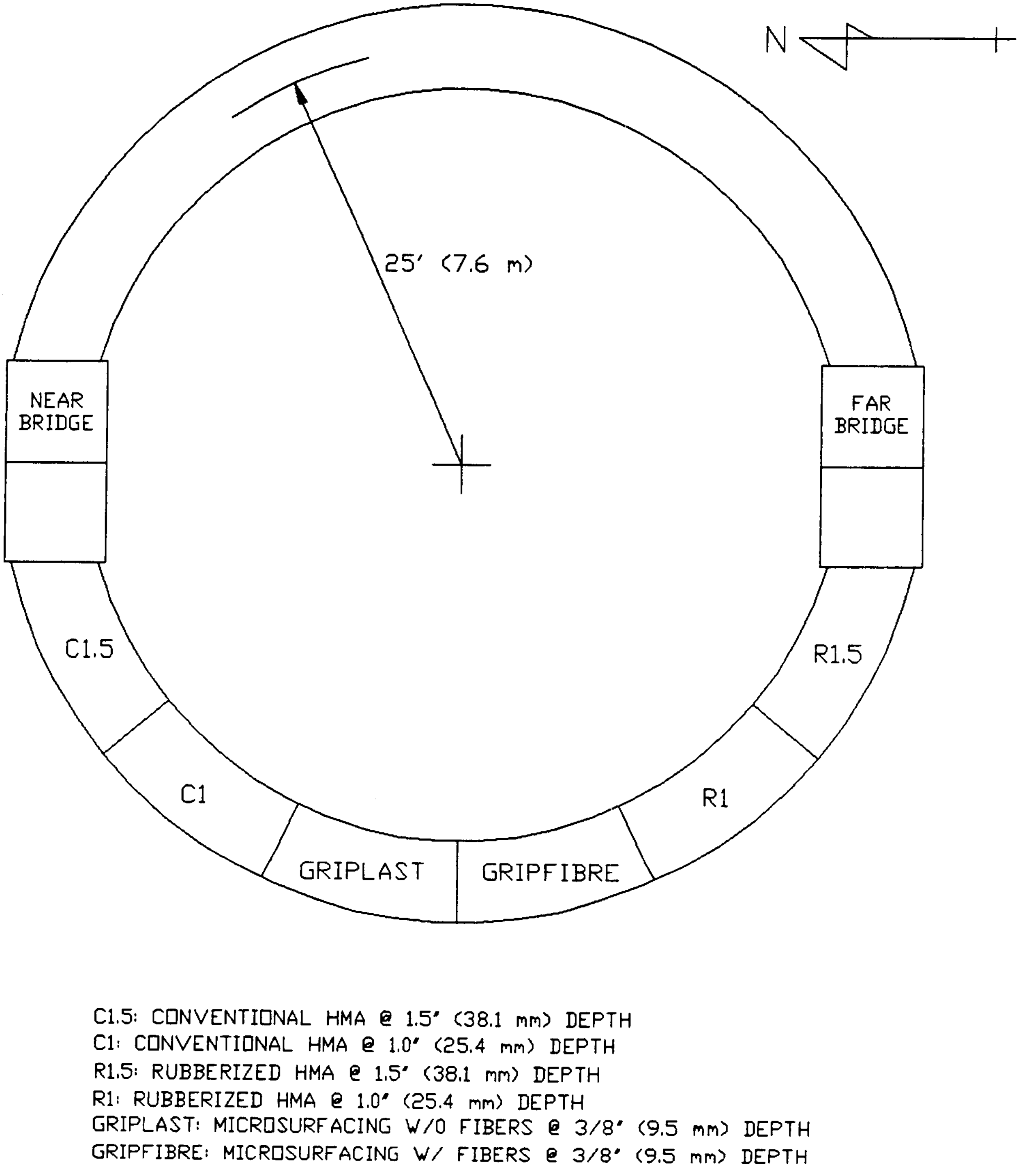
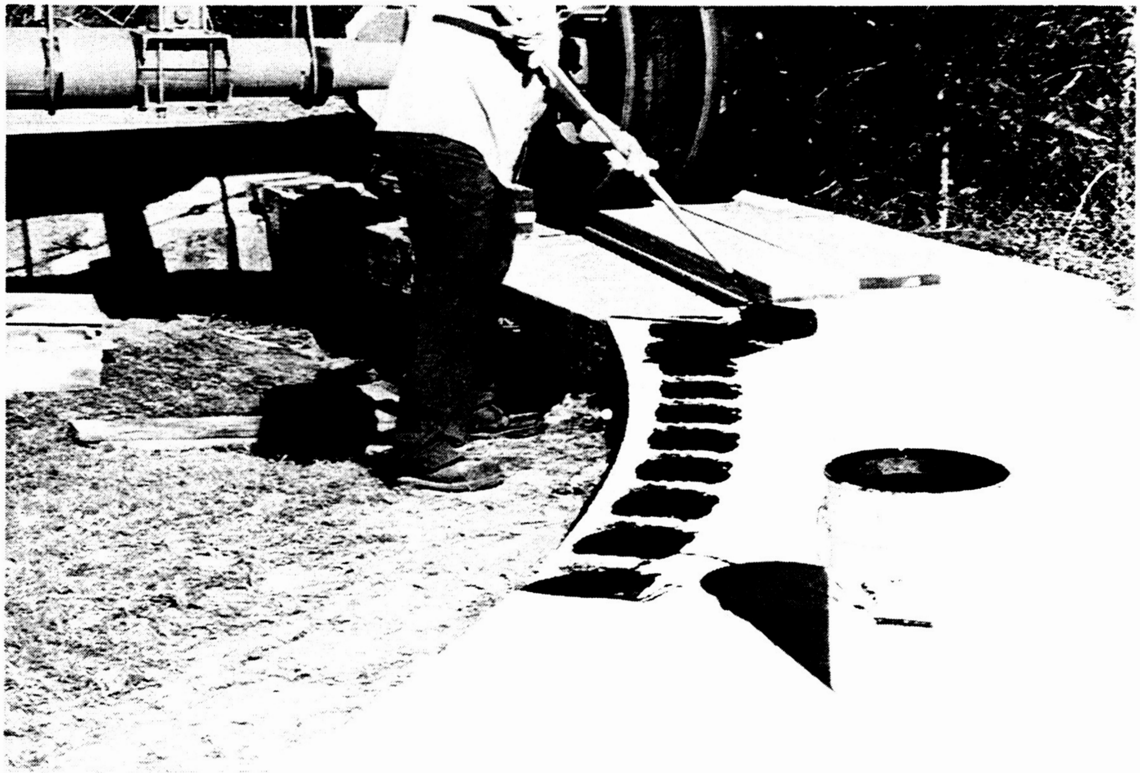
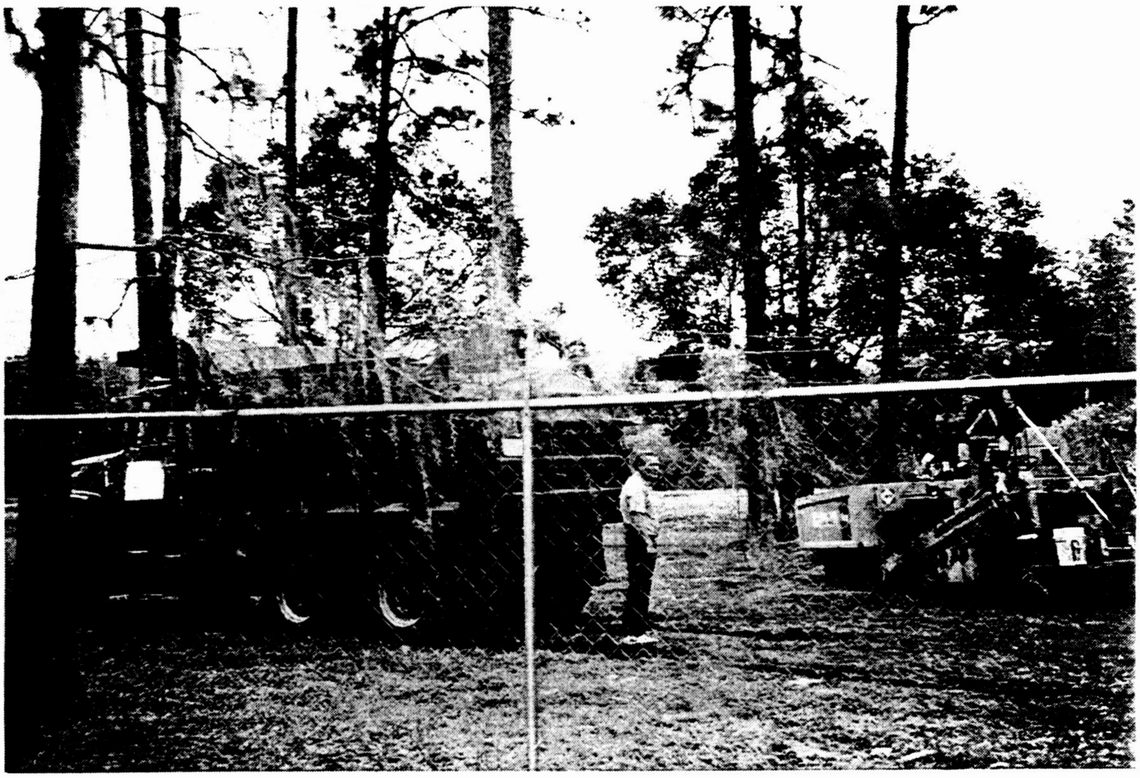


Figure 3: Layout of Pavement Sections for Testing



Photograph 4.4: Application of SS1 tack coat to concrete surface.



Photograph 4.5: Truck load of asphalt concrete mixture.

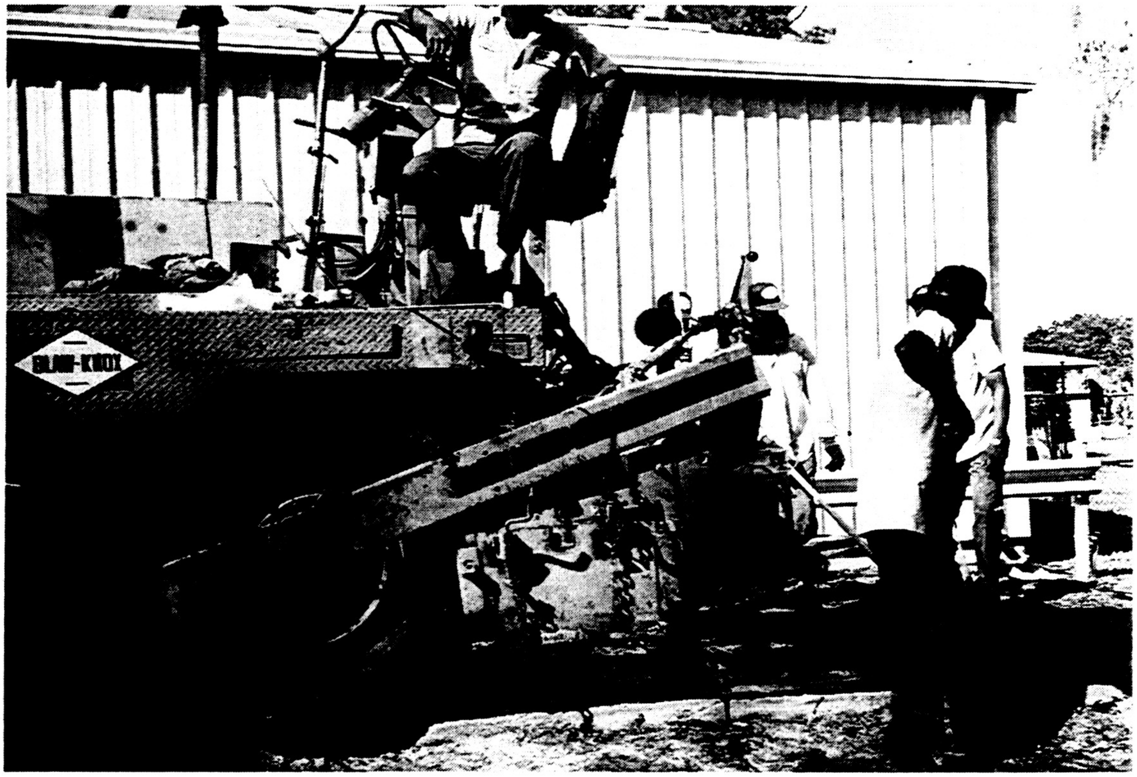
TABLE 2: RUBBER HMA MIX DESIGN (DENSE-GRADED)

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES						
Material	S-3 Stone	Medium Screenings	108A Screenings	Local Sand	JMF	Gradation Design
Blend	40%	20%	20%	20%		Range
3/4 "						
1/2 "	100	100	100	100	100	100
3/8 "	92	100	100	100	97	88 - 98
# 4	40	99	100	100	76	60 - 90
# 10	7	85	90	100	58	40 - 70
# 40	4	43	51	93	39	20 - 45
# 80	3	19	29	23	15	10 - 30
# 200	2	2.5	12	1	5	2 - 6
Sp. Gr.	2.342	2.560	2.420	2.620	2.452	

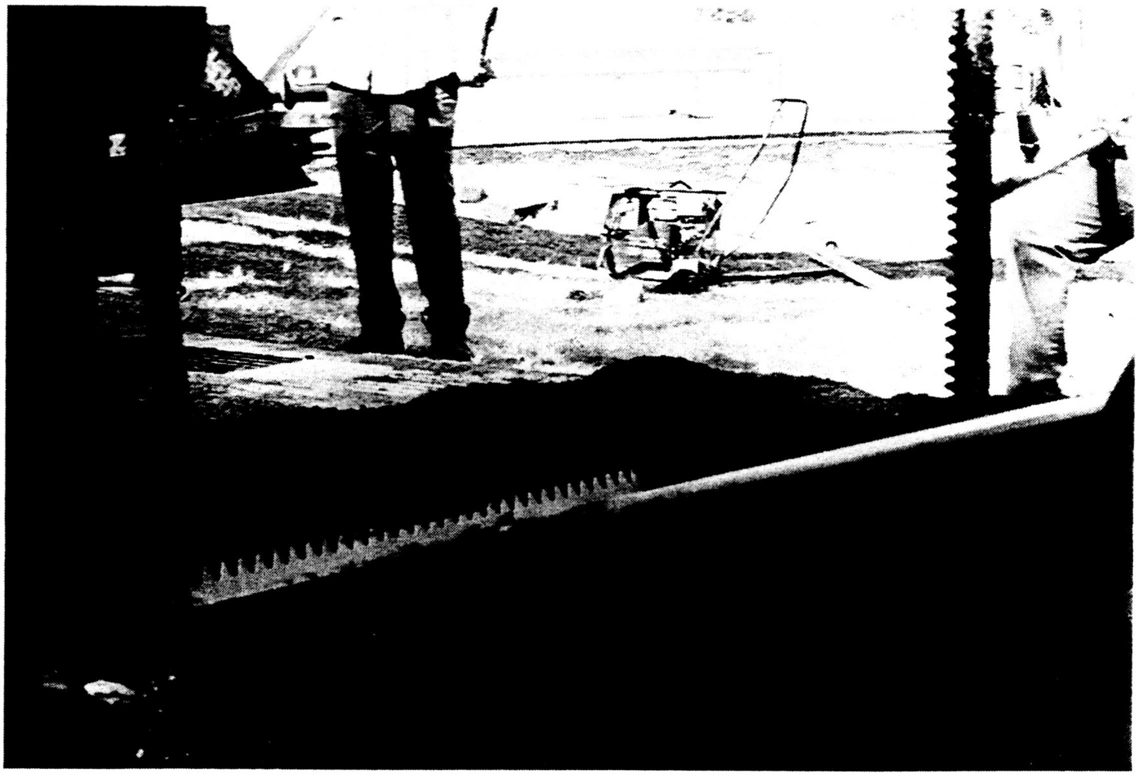
ADDITIONAL DATA	
Asphalt Cement by Wt. of Mix	7.50%
Optimum Asphalt Cement Content	7.14%
Effective Asphalt Cement	6.20%
Bulk Sp. Gr. of Compacted Mix	2.172
Max. Meas. Sp. Gr. of Compacted Mix	2.29
Air Voids	5.20%
V.M.A.	18.10%
Voids Filled	71.30%
Adjusted Stability Averaged	2150
Flow Average	9
Lab Density	135.5 pcf
Mixing Temperature	320 °F
Additives (Antistrip)	0.50%
GTR Content (5 % by wt. of Asphalt Cement)	0.36%

TABLE 3: HMA SPECIMEN TEST DATA

HOT MIX ASPHALT GRADATION			
% Pass Sieve	Conventional S-3	Rubberized S-3 (5% GTR)	JMF S-3
3/4"	100.00	100.00	100.00
1/2"	100.00	99.02	100.00
3/8"	96.67	94.98	97.00
# 4	73.70	74.55	76.00
# 10	54.57	55.66	58.00
# 40	35.78	36.36	39.00
# 80	12.15	12.21	15.00
# 200	3.84	3.81	5.00
ADDITIONAL TEST DATA			
% A/C	7.50	7.50	7.50
Unit Wt.	136.60	135.30	135.50
Stability	1875	2075	Min. 1500
Flow	8.50	9.00	8 to 14
Max. Gr.	2.285	2.312	N/A
Bulk Gr.	2.189	2.168	N/A
Air Voids	4.20	7.30	3 to 7

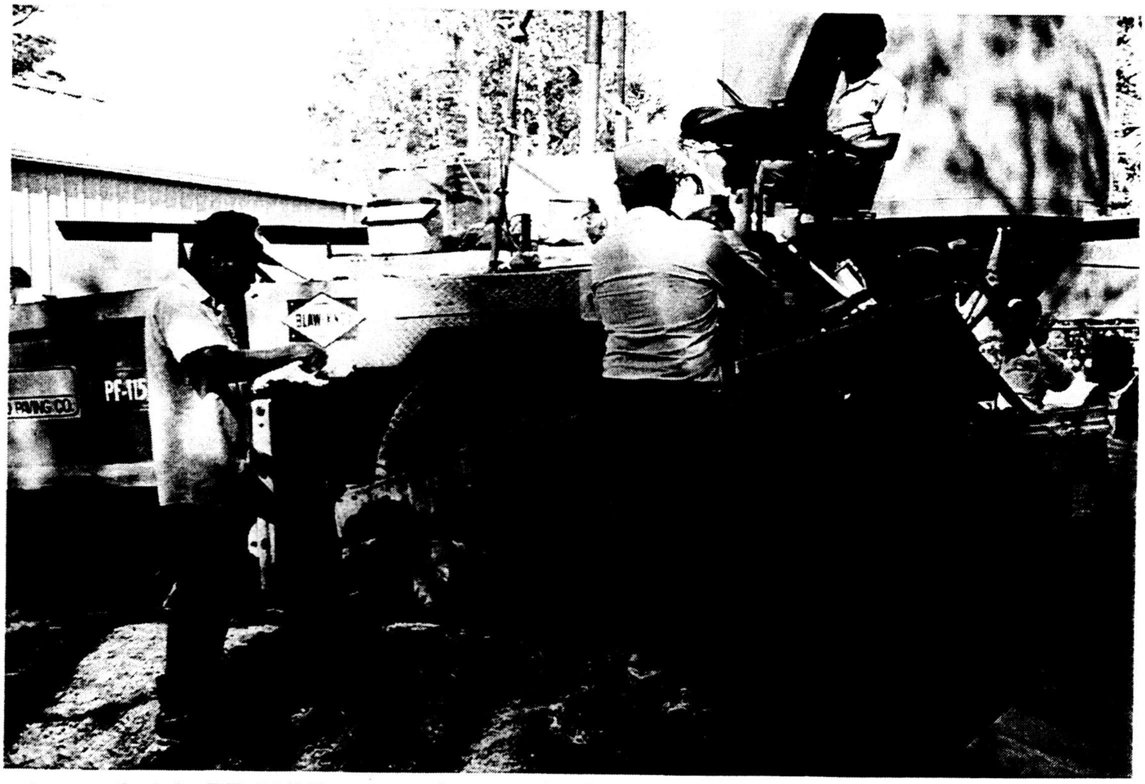


Photograph 4.6: PF-115 Paver and crew placing conventional mix (sections C1.5 & C1).

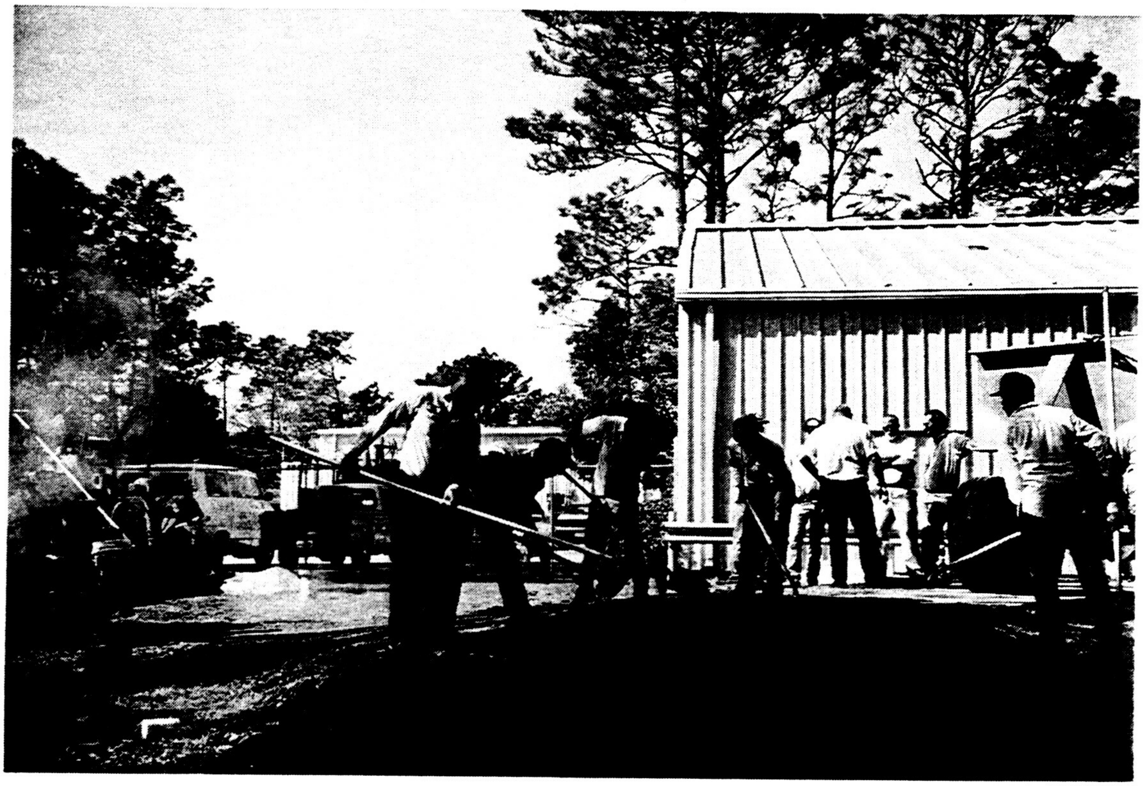


Photograph 4.7: Crew handworking mixtures with rakes, shovels and portable vibrator.

Photograph 4.8: BOMAG Roller compacting conventional mix (sections C1.5 & C1).



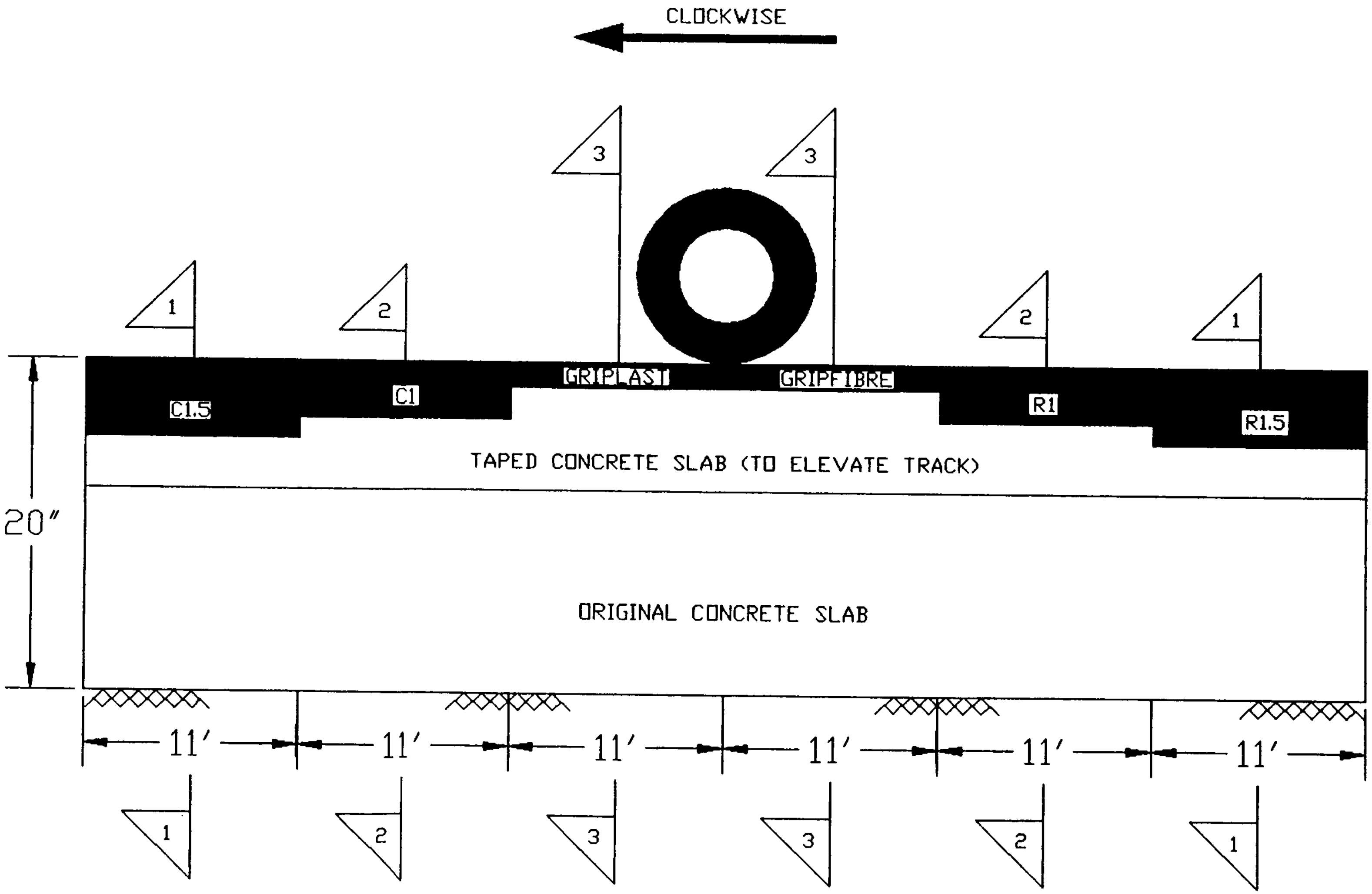
Photograph 4.9: PF-115 Paver and crew placing conventional mix (sections R1.5 & R1).



Photograph 4.10: Crew handworking mixtures with rakes and shovels.

Photograph 4.11: BOMAG Roller compacting rubberized mix (sections R1.5 & R1).

Figure 4: Cross-section Through Centerline of Track Slab and Pavement (1 in = 25.4 mm)



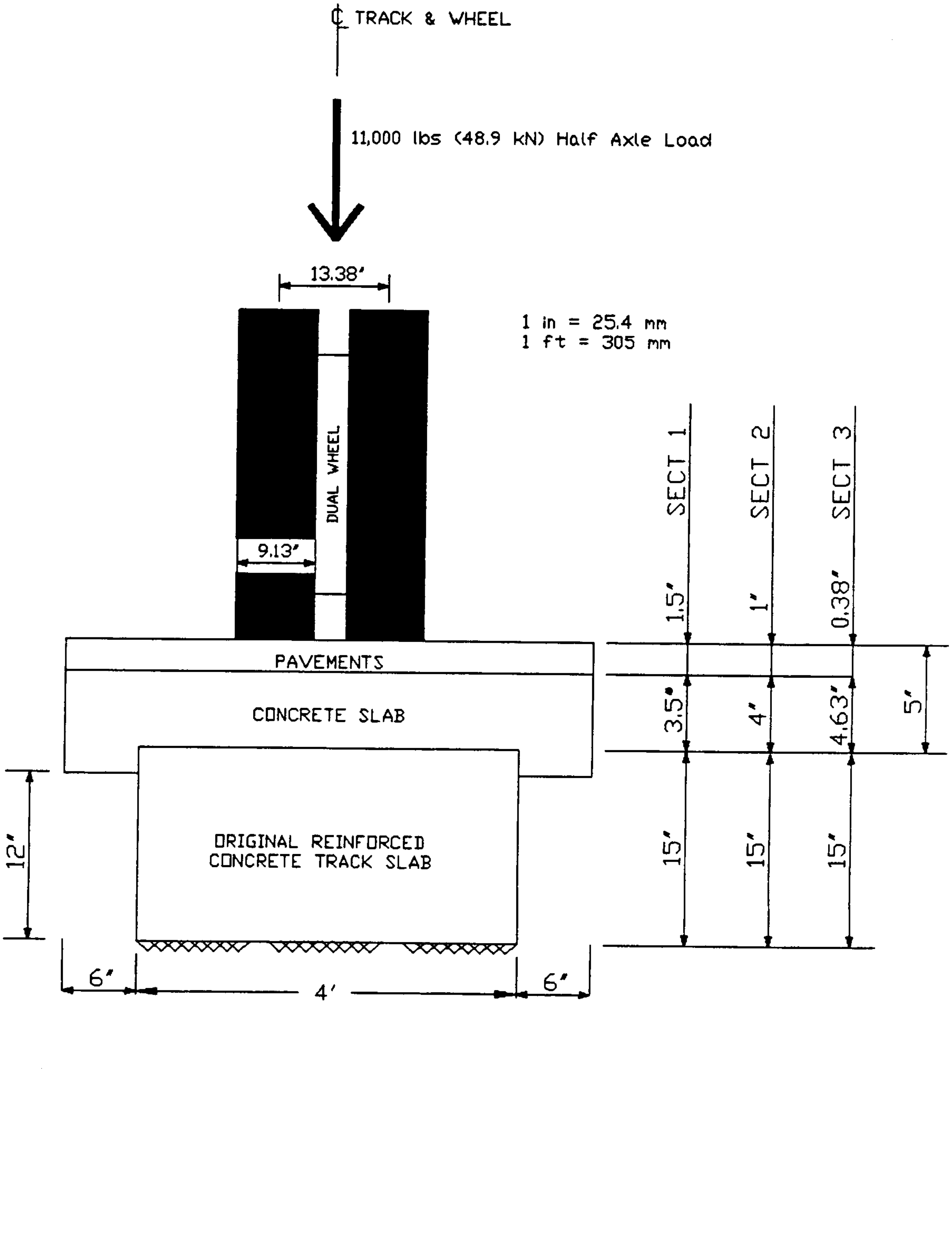


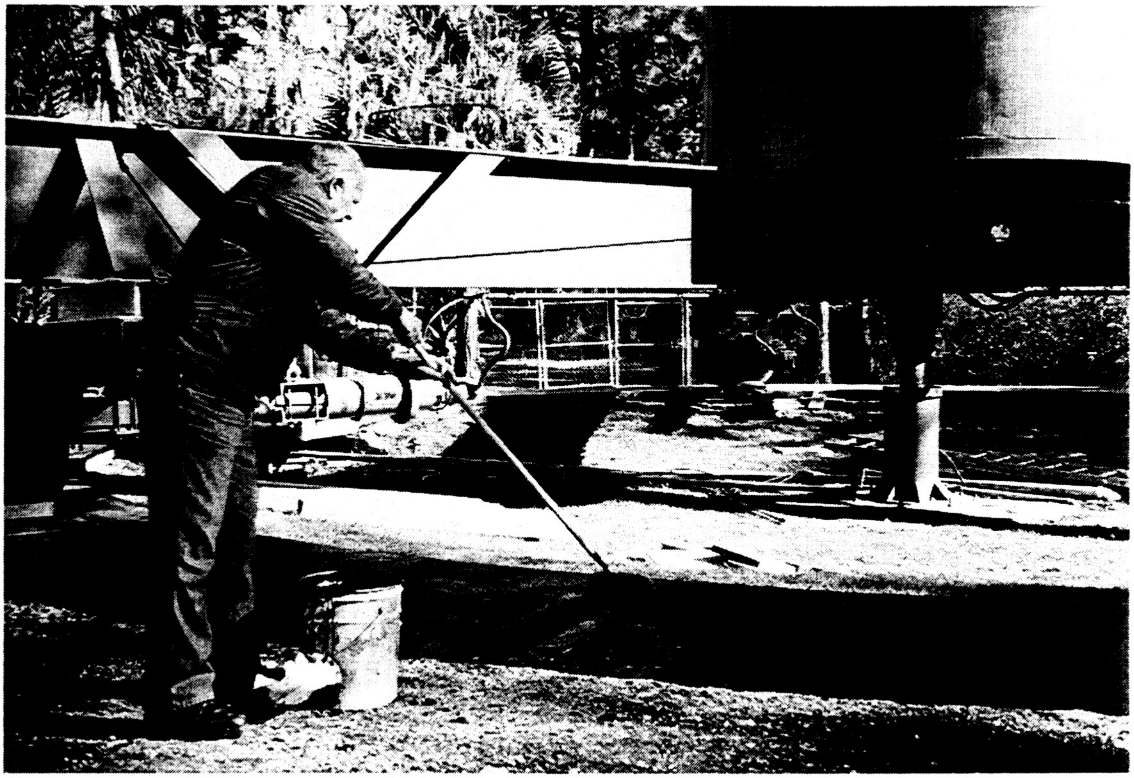
Figure 5: Cross-section of Track Slab and Thickness (see Fig. 4)

MICROSURFACING

Two material mixtures of microsurfacing, **GRIPLAST** (without fibers) and **GRIPFIBRE** (with fibers) were delivered as a Type II mix with 100% aggregate, 12% emulsion, 0.25% to 1.5% mineral filler, and 6% water. Before the mixtures were placed, the concrete surface was cleaned and applied with SS1 tack coat as shown in Photograph 4.12. **GRIPLAST** was laid first by the BERGKAMP continuous mix paver (without the hopper). This dark brown **GRIPLAST** flowed quite easily. Handwork using rakes and burlap was necessary to meet a required uniform thickness. Photographs 4.13 and 4.14 show the sequential procedures.

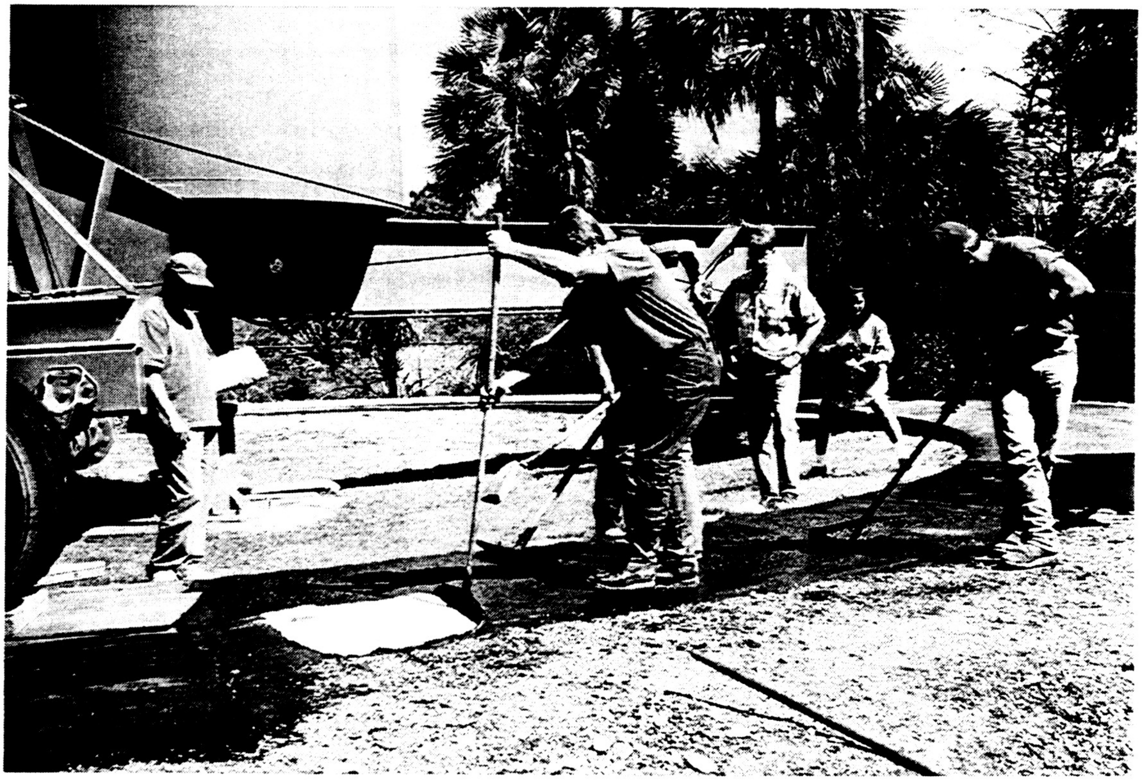
GRIPFIBRE, with an estimated 3 lb/ton (1.5 kg/tonnes), 0.15% by wt. of agg., of polyacrylonitrile fiber in the mix was poured on the section in the same fashion as **GRIPLAST** (see Photograph 4.15).

Both the **GRIPLAST** and **GRIPFIBRE** sections eventually turned black in color, similar to asphalt. Since both sections were completed with handwork, they did not appear to have uniform thickness. On advice from the paving company representative, after curing 1.5 hours, both sections should be smoothed and compacted by slowly rolling the wheels in two passes. This left a light imprint of the tire treads but did not adversely harm the materials at that time. Table 4 provides the microsurfacing mix design data used in this project.



Photograph 4.12: Application of SS1 tack coat to concrete surface (microsurfacing).

Photograph 4.13: BERGKAMP Continuous Mix Paver placing GRIPLAST.



Photograph 4.14: GRIPLAST being handworked with rakes and burlap floated.

Photograph 4.15: GRIPFIBRE being handworked with rake and burlap floated.

TABLE 4: MICROSURFACING MIX DESIGN DATA

AGGREGATE TEST RESULTS GRADATION		
Sieve Size	Type II % Passing	Results % Passing
3/8"	100	100.00
# 4	70 - 100	85.71
# 8	45 - 70	57.63
# 10	******	51.09
# 16	28 - 50	34.16
# 30	15 - 35	21.69
# 50	10 - 25	14.72
# 100	******	10.95
# 200	5 - 15	8.80
TEST	SPEC.	RESULT
Sand Equivalent	65	66.00
Methylene Blue Value	None	2.00
Methylene Blue Factor	None	17.60
Bulk Specific Gravity	None	2.68
Hardness LA Abrasion	< 35%	21.00%
Soundness	< 20 %	12.00%

EMULSION RESIDUE TEST RESULTS		
TEST	SPEC.	RESULT
Penetration, dmm	55 - 90	75.0
Solubility %	97.5 +	99.9
Ductility, 77°F 5 cm/min, cm	40 +	80.0 +
Softening Point, °F	135	141.0

ASPHALT EMULSION TEST RESULTS		
TEST	SPEC.	RESULT
Oil Distilate, by volume %	60 +	63.5
Particle Charge	Positive	Positive
Saybolt Furol Viscosity, SFS	20 - 100	35.0
Sieve Test	0.1 MAX	0.02%
Storage Stability, 1 day	1 % MAX	0.20%

MINERAL FILLER TEST RESULTS SIEVE ANALYSIS		
Sieve Size	Spec. % Passing	Results % Passing
# 30	100	100.00
# 80	95 - 100	98.40
# 200	65 - 100	68.90
Comments: Meets requirements for Type I non-airentrained Portland Cement		

WATER TEST RESULTS		
TEST	UNIT	RESULT
Calcium Hardness	grams/gal	8
Magnesium Hardness	grams/gal	3
Total Hardness	grams/gal	11

JOB MIX RECOMMENDATION	
Aggregate	100%
Mineral Filler	0.25 - 1.5 % of weight of dry aggregate
Asphalt Emulsion	11.75 - 12.25 % of weight of dry aggregate
Water	6.0 % of weight of dry aggregate
Additive	As needed for break control

LAB MIX CHARACTERISTICS	
Set Time	190 sec
Cohesion	
30 min	16.0
60 min	23.0
Open To Traffic	60 min

CHAPTER 5

TEST PROGRAM

A comprehensive interwoven testing and analysis work plan was proposed. Following the literature review of previous work, the research team (professor and graduate students) began the preparation for some laboratory and experimental work. All tasks performed for completion of the research are not duplication of previous efforts by other researchers.

The laboratory program shall investigate the performance criteria of paving mixtures. Like any other construction material, the performance criteria fall under two major parameters -- strength and durability of the material. Under the strength criterion, the optimum percent additive in the mixture to gain the maximum shear and tensile strength, stability and modulus values. Durability of the material is resulted primarily by fatigue life. Other factors involving the temperature and moisture will also be investigated.

Asphalt Concrete Mixtures

The asphalt concrete pavement mixture used for the conventional pavement section in the test was according to FDOT or NASA specification and was supplied by a local paving company. Few samples of mixtures with additives were tried initially according to the ingredient available in the literature. Optimum design of the additives may be based on the results of the performance tests.

Laboratory Testing of Mix Specimen

One of the objectives of laboratory testing is to attempt to correlate material characterization determined in the laboratory to the performance in the field. Some of the characteristics of the mixture are density, stability, shear and tensile strength, and stiffness modulus of the mix. These values are normally available from the literature. However, if significant changes of mix design are needed for the best performance, the laboratory testings are required. All test procedures will be according to ASTM specifications. The paving company, using their own facilities, performed two laboratory tests on four asphalt specimens (two conventional, and two rubberized). These tests were the Marshall Stability (ASTM D 1559) and the Bulk Specific Gravity and Density (ASTM D 2041), giving the results shown in Table 3. In addition, the paving company determined the in-place asphalt densities using a 3411-B Troxler Nuclear Density Gage. For the microsurfacing, the paving company supplied an independent report on mix design and test data, (see Table 4).

Performance Tests Using UCF Circular Accelerated Test Track (UCF-CATT) Testing Facility

The pavement test sections with and without additive mixture were placed on the test track (UCF-CATT). Wheel load, velocity, and the number of load applications were recorded from data acquisition system in the test facility. The total number of load applications was converted in terms of the service life of year if ADT was assigned for its respective highway section.

Analysis Work

Along with the laboratory and performance test, the analyses are also proposed to further evaluate the performance of each test section. For example, the deformation/compressive strain versus the number of load applications for each pavement section will be plotted. Pavement analysis program such as **CHEVRON** for flexible pavement and other programs will also be searched for analytical work to compare with experimental test results.

Cost Effectiveness and Life Cycle Cost Evaluation

To determine the cost-effectiveness of additive in the mixture, the life of pavement service must be known. There are many factors that affect the life of pavement section, such as environment, traffic, quality of construction and materials, and even construction techniques. It is difficult to assess the cost-effectiveness of each additive mixture. However, the unit cost of materials and labors shall be easily estimated. Overall life-cycle cost analysis will be based upon (1) the cost of initial material and construction labor and (2) the extension of service life of the load applications from test facility. Only the materials with the most potential identified in the proposed study will be used in the life-cycle cost analysis.

Development of Specifications

From the results of this research study, the specification will be developed for the optimum content of additive for different type of mixtures, physical and chemical requirements, material controls, equipment requirement. However, all pavement sections failed in very early stages and the data obtained from the test results are considered not adequate for the development of specifications.

CHAPTER 6

PERFORMANCE TESTING RESULTS

Pavement performance is an important factor of pavement design, rehabilitation, and management since it provides a framework upon which a judgement on the success can be made on a design procedure, or on a need for further improvements **[14]**. Included in pavement performance are distress, evaluation, serviceability, skid-resistance, and non-destructive testing. This research study only focuses on the pavement distress and evaluation through test results.

Distress can be divided into structural failure and functional failure. The structural failure is associated with the ability of the pavement to carry the design load while the functional failure concerns the pavements riding comfort and safety. Structural failures may end up in functional failures, whereas functional failures may be structurally sound. Distress is further broken down into load-associated, and non-load-associated, which may be caused by climates, materials, or construction. A non-load-associated distress may later become more severe as the number of load repetitions increases.

Among the types of distress of concern in this report are:

1. **Alligator or fatigue cracking**, a structural load-associated failure, which is a series of interconnecting cracks caused by failure of asphalt surface or stabilized base under repeated traffic loading.

2. **Block cracking**, a structural non-load-associated failure, which divides the asphalt surface in large rectangular pieces and is caused by the shrinkage asphalt temperature cycling; occurrence indicates the asphalt has significantly hardened.

3. **Longitudinal and transverse cracking**, a structural non-load-associated failure, which is caused by the shrinkage of asphalt due to low temperatures or asphalt hardening or from reflective cracks beneath the asphalt surface which includes cracks in slabs away from joints.

4. **Bleeding**, a functional non-load-associated failure, which is a surface film of bituminous material creating a shiny reflecting surface which becomes sticky; caused by a high asphalt content or low air void content.

5. **Corrugation**, a functional non-load-associated failure, which is a form of plastic movement typified by ripples across the asphalt surface and is a result of shear action on the pavement surface or between pavement surface and the base material.

6. **Joint reflection cracking**, a structural non-load-associated failure, which occurs at both transverse and longitudinal joints in the underlying concrete slab and caused by movement of the slab from thermal or moisture changes.

7. **Rutting**, a functional load-associated failure, which is a depression in the wheel paths but may cause uplift of the asphalt surface along the sides of the rut, and stems from the permanent deformation in the pavement layers. Usually caused by consolidation or lateral movement of the materials under loading, and plastic movement in hot weather or inadequate compaction.

8. **Polished aggregate**, a functional load-associated failure, which a portion of the aggregates extending above the asphalt surface is either very small or without rough or angular particles to provide adequate skid-resistance; occurs in the wheel path from repeated loads and tire abrasion.

9. **Raveling and weathering**, a functional non-load-associated failure, which are the wearing away of the pavement surface caused by the dislodging of aggregate particles due to stripping and the loss of asphalt binder.

Using the UCF-CATT facility, performance testing started on **3/12/96** and ended on **4/18/96**, with a time span of approximately five weeks. However, all the sections were only tested for 57.25 daytime hours (see Table 5 for Test Track Log data). This time difference was due to a problems on bridge ramps and deteriorated pavement sections which caused testing to be suspended.

The above start time gave asphalt test sections (**C1.5**, **C1**, **R1.5**, **R1**) a month, and the microsurfacing test sections eight days to properly cure and set before testing started. The three dual-wheel, each carrying 11 kips (48.9 kN) and a tire pressure of 110 psi (758.4 kPa), were rotated in a clockwise direction at an operating speed of approximately 10 mph (4.5 m/s). Up to 9 hours of daily operation included pauses after approximately two hours of operation for track maintenance and/or photographs. During this time period, outdoor temperatures ranged between 60° F (15.5°C) and 80° F (26.7°C). All pavement sections were tested while water was constantly sprayed on the track at two locations, adjacent to each bridge, to reduce excessive wearing and heating of the tires. All pavement sections were subjected to a total of 58,215 load repetitions.

Throughout this testing, the pavement distress was documented with photographs taken when one or more of the above nine failures occurred. The photos were used to discuss the test results for each of the six pavement sections tested.

TABLE 5: TEST TRACK LOG

DATE	START TIME (hr)	END TIME (hr)	TOTAL TIME (hr)	START MILE (mile)	END MILE (mile)	TOTAL MILE** (mile)	REP*	TOTAL REP	Wheel Velocity (mph)
3-12	9.00	9.33	0.33	2.7	6.0	3.3	333	333	10.0
3-18	9.00	10.15	1.15	6.0	17.5	11.5	1160	1493	10.0
3-18	13.57	14.73	1.17	17.5	29.2	11.7	1180	2672	10.0
3-19	10.75	15.82	5.07	29.2	78.9	49.7	5012	7684	9.8
3-20	7.37	8.87	1.50	78.9	93.6	14.7	1482	9167	9.8
3-21	11.67	16.26	4.59	93.6	138.7	45.1	4548	13714	9.8
3-22	7.28	14.33	7.05	138.7	209.6	70.9	7150	20864	10.1
3-25	6.90	14.44	7.54	209.6	286.5	76.9	7755	28619	10.2
3-26	8.62	14.29	5.67	286.5	342.9	56.4	5687	34306	9.9
4-15	7.02	16.02	9.00	342.9	435.7	92.8	9358	43664	10.3
4-16	13.17	16.47	3.30	435.7	468.9	33.2	3348	47012	10.1
4-17	7.02	15.66	8.64	468.9	558.8	89.9	9066	56077	10.4
4-18	7.22	9.44	2.22	558.8	580.0	21.2	2138	58215	9.5
Totals			**57.25**			**577.3**		**58215**	

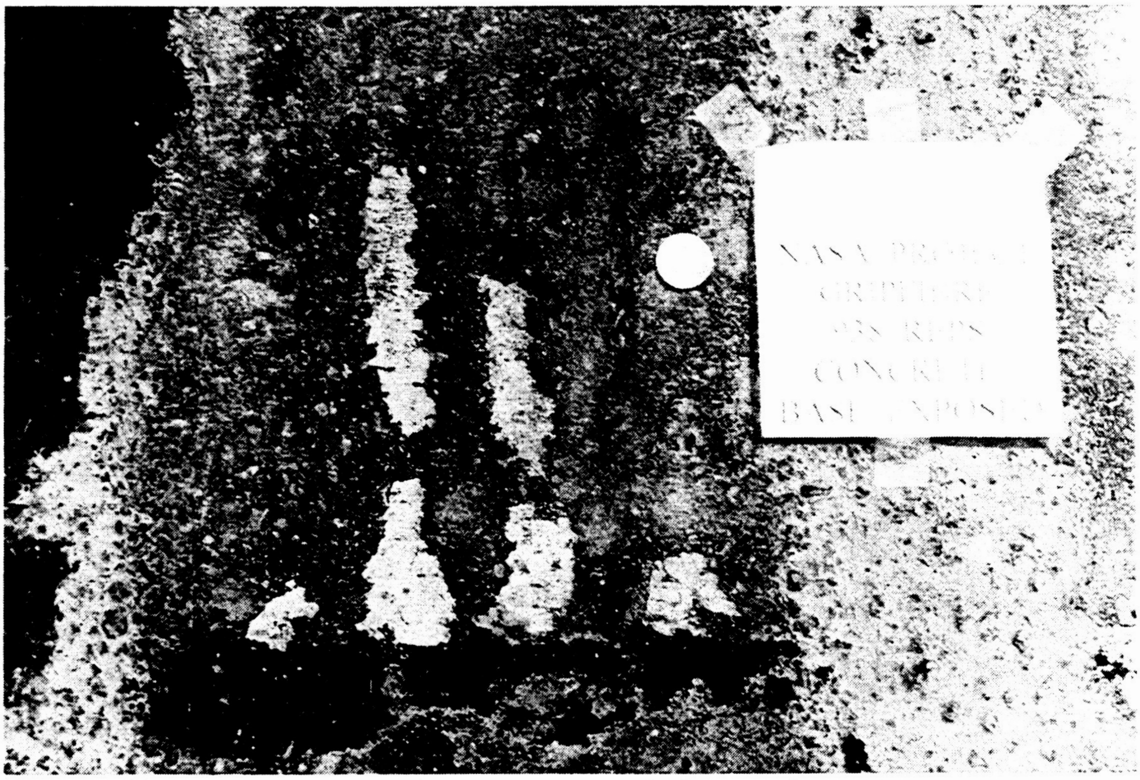
* Reps = miles x 100.84
** Mileage from hubometer (nearest 0.10 mile)
- Clockwise Rotation
- 1 mile = 1.6 km
- 1 mph = 1.6 km/h

GRIPFIBRE

Photographs 6.1 and 6.2 show the early problems of the pavement section after approximately one hour (938 reps) of testing. The photos exhibit that the material has been worn off and rutted, printed with tire tread, and that the concrete slab underneath has been exposed. Deterioration started under the inner wheel path and then extended to the outer wheel path (after approximately 2.6 hour or 2,673 reps). After approximately six hours (6,161 reps), at other locations the material continues to be worn off and rutted under both wheel paths as shown in Photograph 6.3. The distress worsens, exposing more concrete slab as shown in Photograph 6.4, as well as ravelling and aggregate polishing after approximately 21 hours (20,864 reps). From Photograph 6.5, after almost 35 hours (34,306 reps) of testing, large areas of the pavement section were completely worn off underneath the wheel paths. Also, the photo shows material uplift between the dual-tires and along the outside edge of each tire. After nearly 46 hours (46,135 reps) of testing the paving material was clearly uplifted, ravelled, rutted, and the aggregate polished as shown in Photograph 6.6.

GRIPLAST

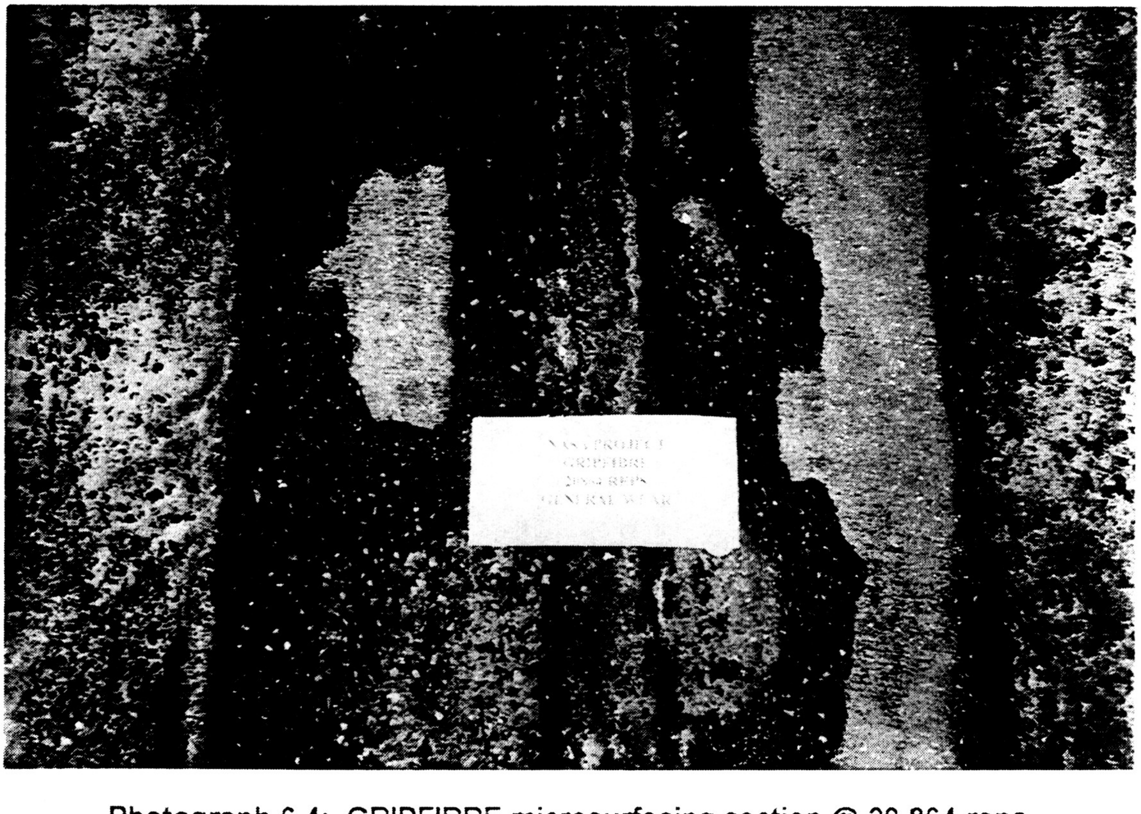
The GRIPLAST performed the same problems in the early stages, as with GRIPFIBRE (see Photographs 6.7 and 6.8). After approximately 7.6 hours (7,684 reps), the material continues to be worn off and rutted under both wheel paths at other locations as shown in Photograph 6.9. The distress even worsens after approximately 29 hours (28,890 reps) (see Photograph 6.10). Photograph 6.11 displays the same distress as Photograph 6.6 of GRIPFIBRE.



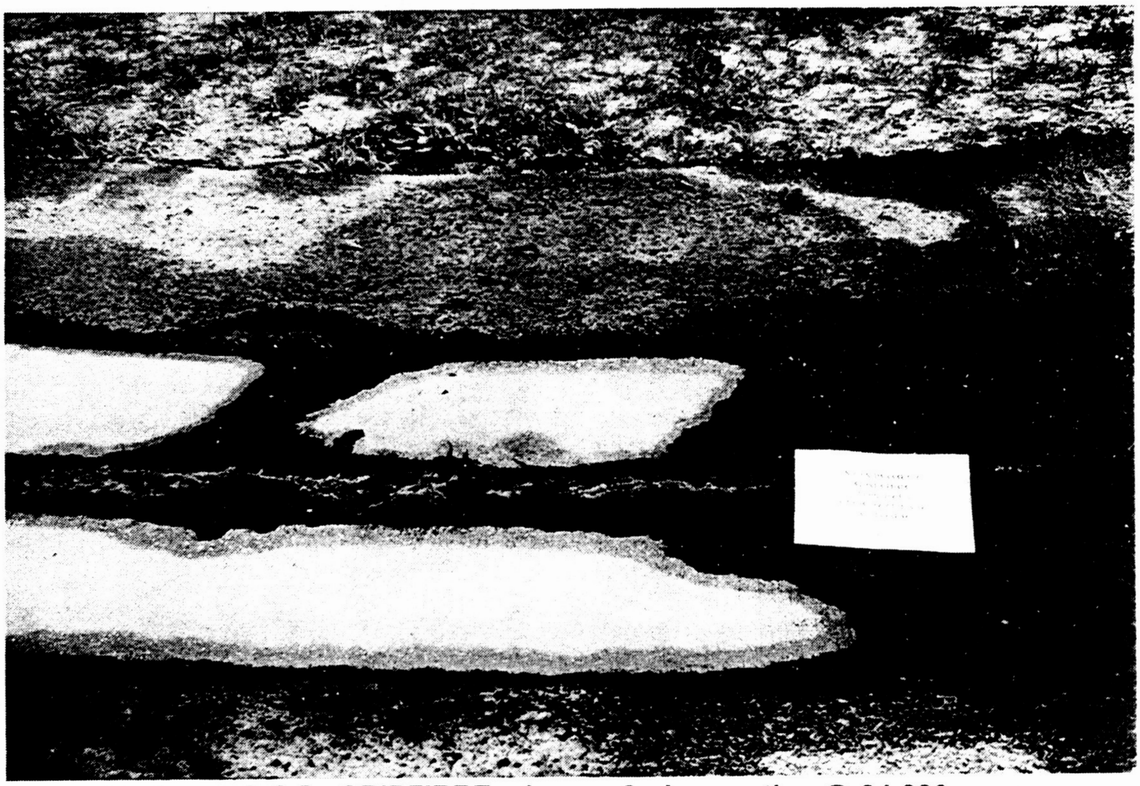
Photograph 6.1: GRIPFIBRE microsurfacing section @ 938 reps

Photograph 6.2: GRIPFIBRE microsurfacing section @ 2,673 reps

Photograph 6.3: GRIPFIBRE microsurfacing section @ 6,161 reps

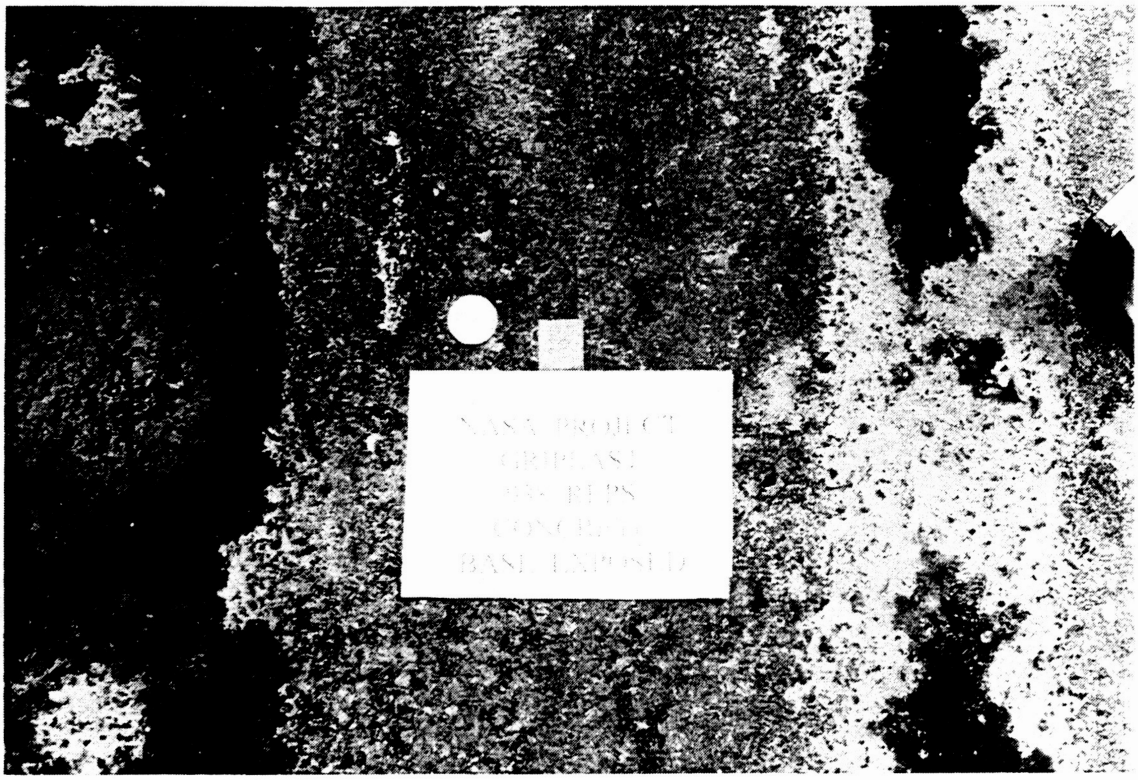


Photograph 6.4: GRIPFIBRE microsurfacing section @ 20,864 reps

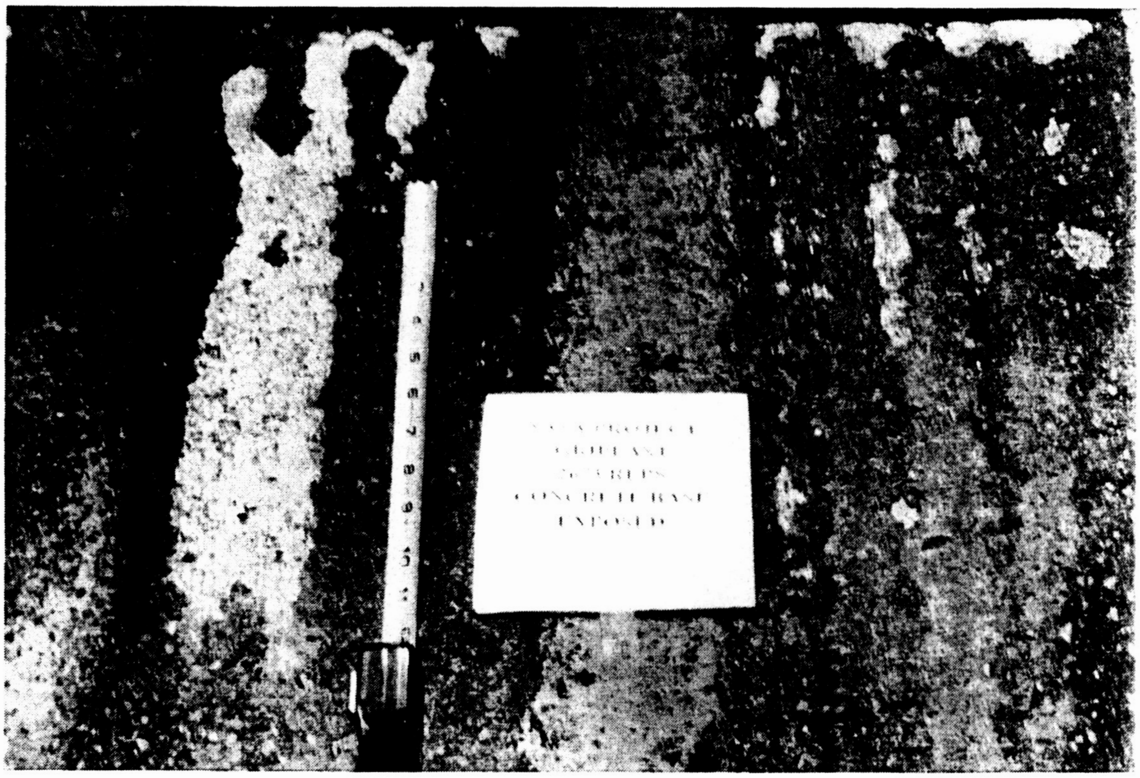


Photograph 6.5: GRIPFIBRE microsurfacing section @ 34,306 reps

Photograph 6.6: GRIPFIBRE microsurfacing section @ 46,135 reps

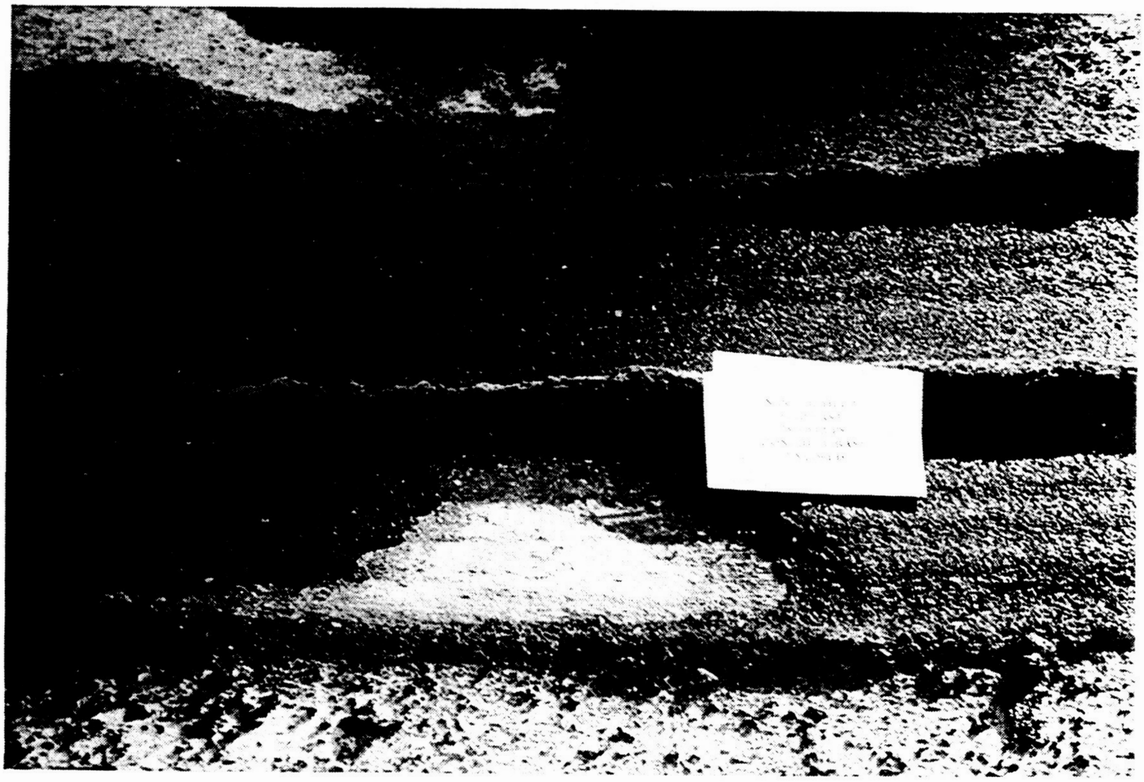


Photograph 6.7: GRIPLAST microsurfacing section @ 938 reps



Photograph 6.8: GRIPLAST microsurfacing section @ 2,673 reps

Photograph 6.9: GRIPLAST microsurfacing section @ 7.684 reps



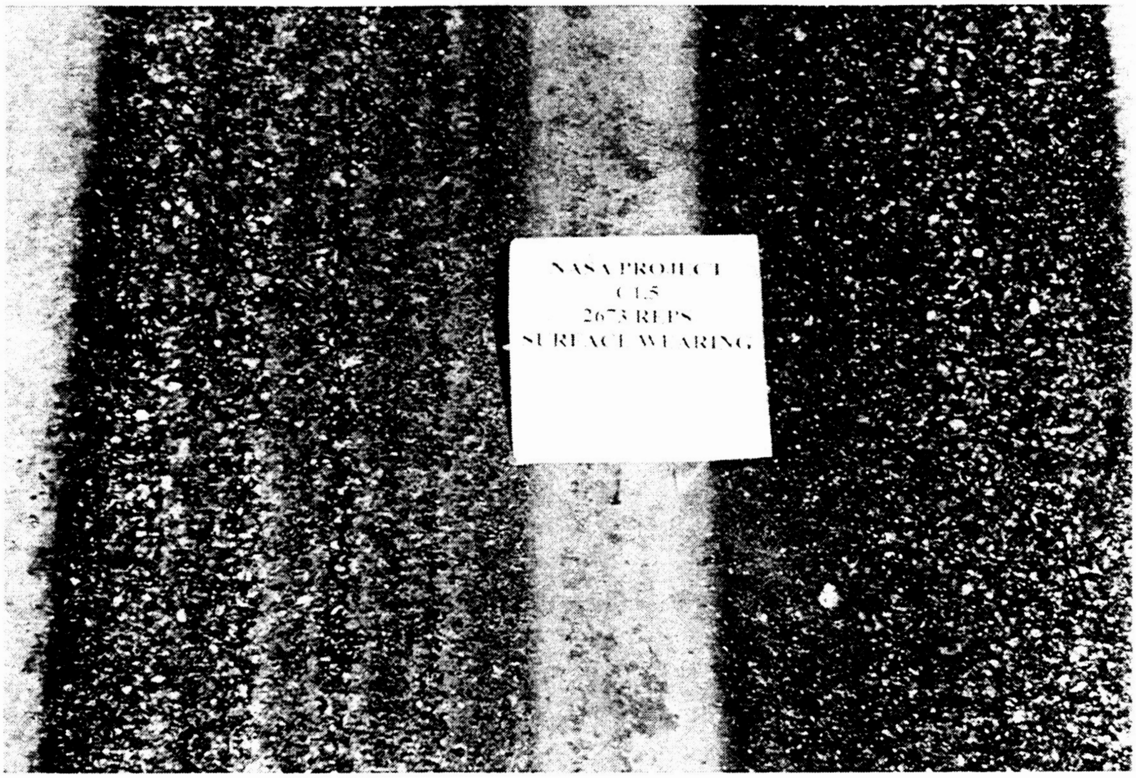
Photograph 6.10: GRIPLAST microsurfacing section @ 28,890 reps

Photograph 6.11: GRIPLAST microsurfacing section @ 46,135 reps

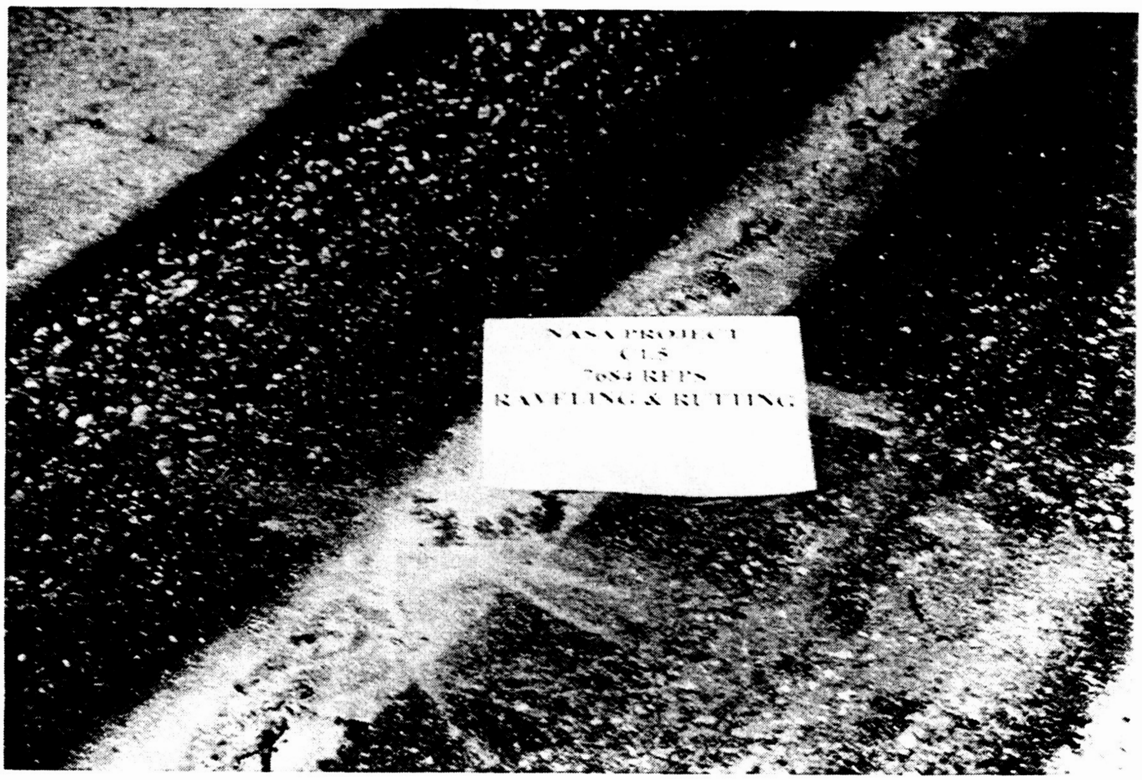
CONVENTIONAL ASPHALT

For the **C1.5** pavement section, after nearly 2.6 hours (2,673 reps) of testing shows some aggregates polished (Photograph 6.12) under both wheel paths. After approximately 7.6 hours (7,684 reps), the material experiences more polished aggregate with some ravelling and rutting, largely under the inner wheel path as seen in Photograph 6.13. Photograph 6.14 indicates more of the same after almost 29 hours (28,890 reps), except that at this location more distress occurs under the outer wheel path. Photograph 6.15 indicates worsening polished aggregate, ravelling, deeper rutting and material uplift at the tire edges especially under the outer wheel path after 46 hours (46,135 reps) of testing.

For the **C1** pavement section, after 7.6 hours (7,684 reps) of testing some polished aggregates were revealed under both wheel paths (Photograph 6.16). Photograph 6.17 shows more polished aggregate as well as rutting under the outer wheel path after almost 21 hours (20,846



Photograph 6.12: Conventional section C1.5 @ 2,673 reps

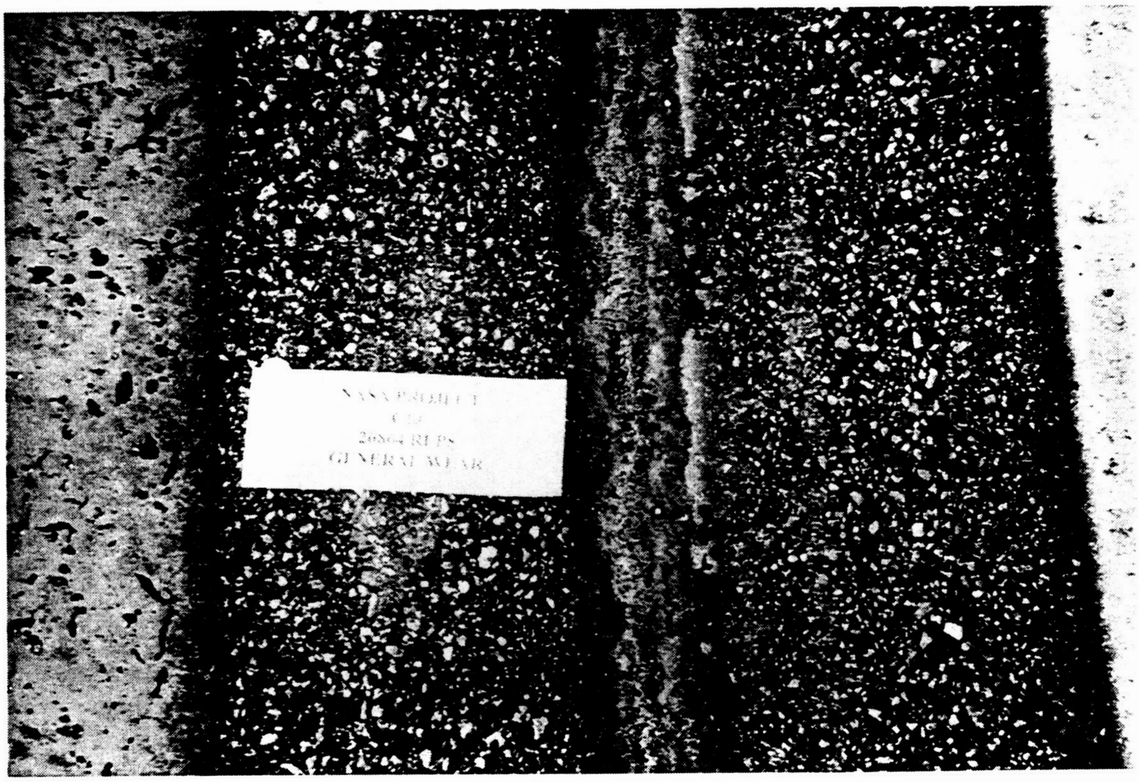


Photograph 6.13: Conventional section C1.5 @ 7,684 reps

Photograph 6.14: Conventional section C1.5 @ 28,890 reps

Photograph 6.15: Conventional section C1.5 @ 46,135 reps

Photograph 6.16: Conventional section C1 @ 7,684 reps



Photograph 6.17: Conventional section C1 @ 20,864 reps

reps). After nearly 46 hours (46,135 reps) the distress worsened, and further ravelling, rutting, and material uplift at the tire edges occurred (see Photograph 6.18).

RUBBERIZED ASPHALT

R1.5 pavement displayed almost the same distresses as **C1.5** during the course of testing. Photographs 6.19 through 6.25 show the similar progress of distress as in Photographs 6.12 through 6.15.

For the **R1** pavement section, after nearly 7.6 hours (7,684 reps) Photograph 6.24 indicates a location of some ravelling and rutting under both wheel paths. Photograph 6.25 shows polished aggregate under both wheel paths and rutting under the outer wheel path after nearly 21 hours (20,864 reps). Photograph 6.26 shows a worsened polished aggregate and ravelling occurring under the inner wheel path after almost 29 hours (28,890 reps) of testing. After approximately 34 hours (34,306 reps) Photograph 6.27, which is mislabeled in the photo, shows a location 4"x12" (10.2 cm x 30.5 cm) of severe ravelling and rutting such that the concrete base is exposed under the outer wheel path. Photograph 6.28 is the same location as Photograph 6.27, except that the 4"x12" (10.2 cm x 30.5 cm) area had been patched with an epoxy-type material, it indicates that after approximately 46 hours (46,135 reps) of further testing more concrete base was exposed 4"x10" (10.2 cm x 25.4 cm) on both sides of the patch under the outer wheel path. For the same time period of 46 hours Photograph 6.29 shows another location where polished aggregate, ravelling, and rutting occurred. After almost 52.5 hours (52,578 reps) Photograph 6.30 shows the same location of photo 6.27 except an area 8"x18" (20.3 cm x 45.7 cm)under the inner wheel path where the concrete base is exposed. Photograph 6.31 indicates another location 3"x9" (7.6 cm x 22.8 cm) where the concrete base is exposed under the outer wheel path as well as rutting and material uplift.

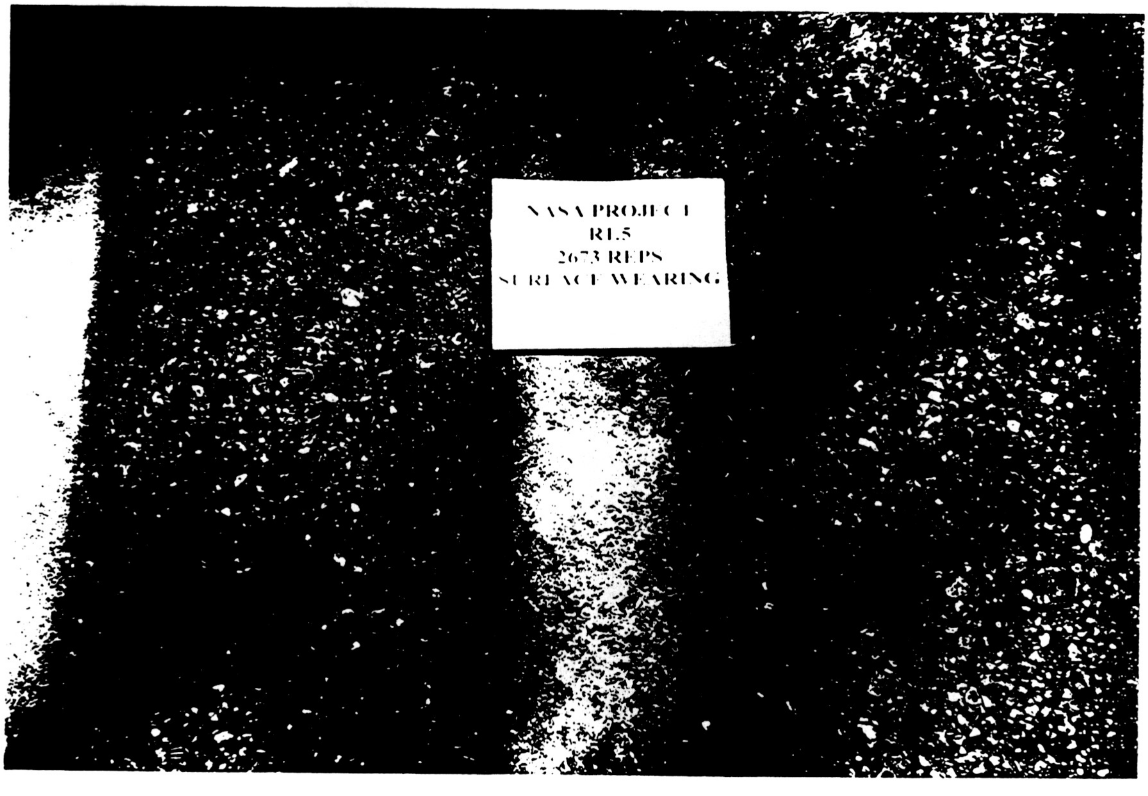
During the five weeks of testing, the first five pavement distresses as listed in previous section seemed not to occur. However, a joint reflection crack on the microsurfacing sections occurred at the concrete track slab construction joint, (refer to Figure 2 for joint location).

In addition to pavement distress by photo documentation, rut depths were measured using a transit/level and a metal graduated tape. Rut depths are measured at the center of each wheel path and between the tires to nearest 1/32" (0.08 cm). The locations (plan) of the measurements are shown in Figure 6 and the rut depth data are presented in Table 6.

Measured rut depths after 58,215 load repetitions are shown in Table 6 for the centers of each wheel path and between the tires. For section **R1.5** rut depths range from 0.104" (2.6 mm) to 1.479" (37.5 mm) with an average of 0.528" (13.4 mm) in the wheel paths with a 0.203" (5.2 mm) uplift between the tires. For section **R1** depths range from 0.177" (4.5 mm) to 1.0" (25.4 mm) with an average of 0.609" (15.5 mm) in the wheel paths with a 0.344" (8.7 mm) uplift between the tires. For **GRIPFIBRE** section a range from 0.271" (6.9 mm) to 0.344" (8.7 mm) with an average of 0.304" (7.7 mm) in the wheel paths with 1.719" (43.6 mm) uplift between the tires. For **GRIPLAST** section a range from 0.198" (5 mm) to 0.438" (11.1 mm) with an average of 0.313" (8 mm)in the wheel paths with 1.39" (35.3 mm) uplift between the tires. For section **C1** depths range from 0.156" (4 mm) to 0.375" (9.5 mm) with an average of 0.254" (6.5 mm) in the wheel paths with 0.422" (10.7 mm) uplift between the tires. For section **C1.5** rut depths range from 0.156" (4 mm) to 0.646" (16.4 mm) with an average of 0.38" (9.7 mm) in the wheel paths with 0.375" (9.5 mm) uplift between the tires.

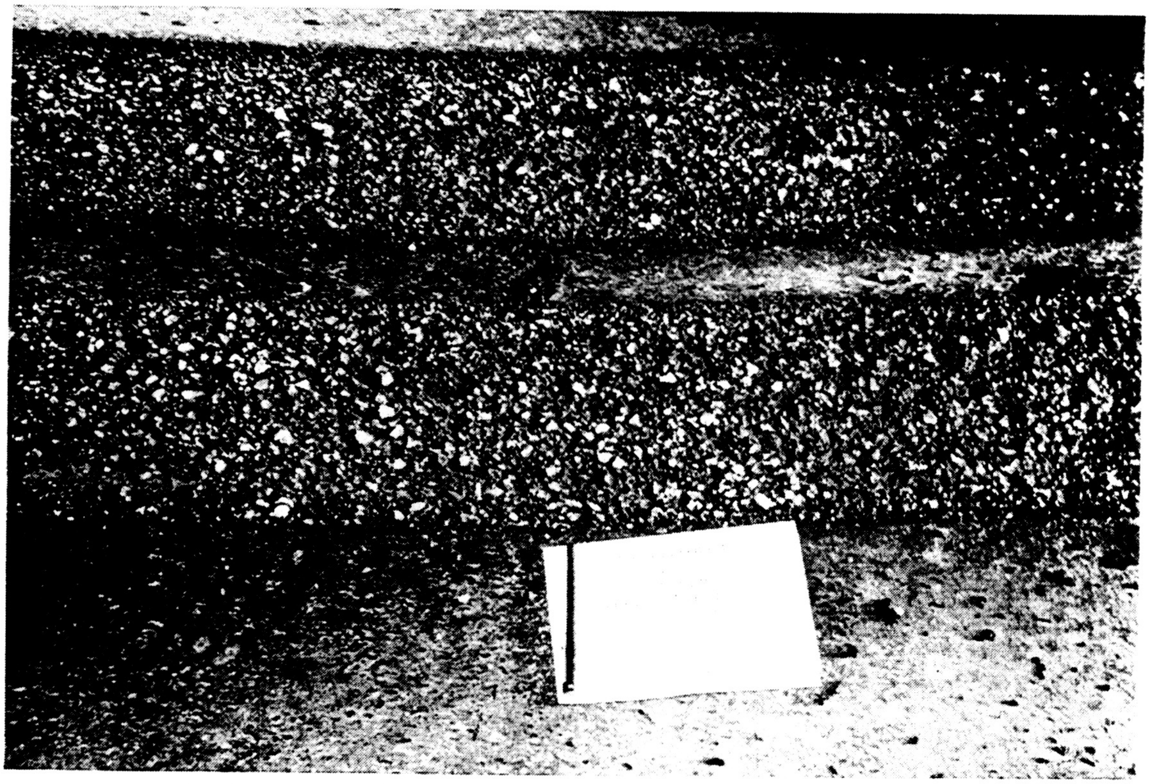
After 58,215 load repetitions and 57.25 hours of testing, Figure 7 indicates the areas of pavement material completely worn away. For the 1.5" (38.1 mm) thick **R1.5** pavement section 9.5% of its volume is worn away, for the as thin as 0.86" (21.8 mm) thick **R1** section 29.6% was worn away, for the **GRIPFIBRE** section 66.3% is worn away, for the **GRIPLAST** section it is 58.7%, and for both pavement sections **C1** and **C1.5** no concrete base is exposed.

Photograph 6.18: Conventional section C1 @ 46,135 reps



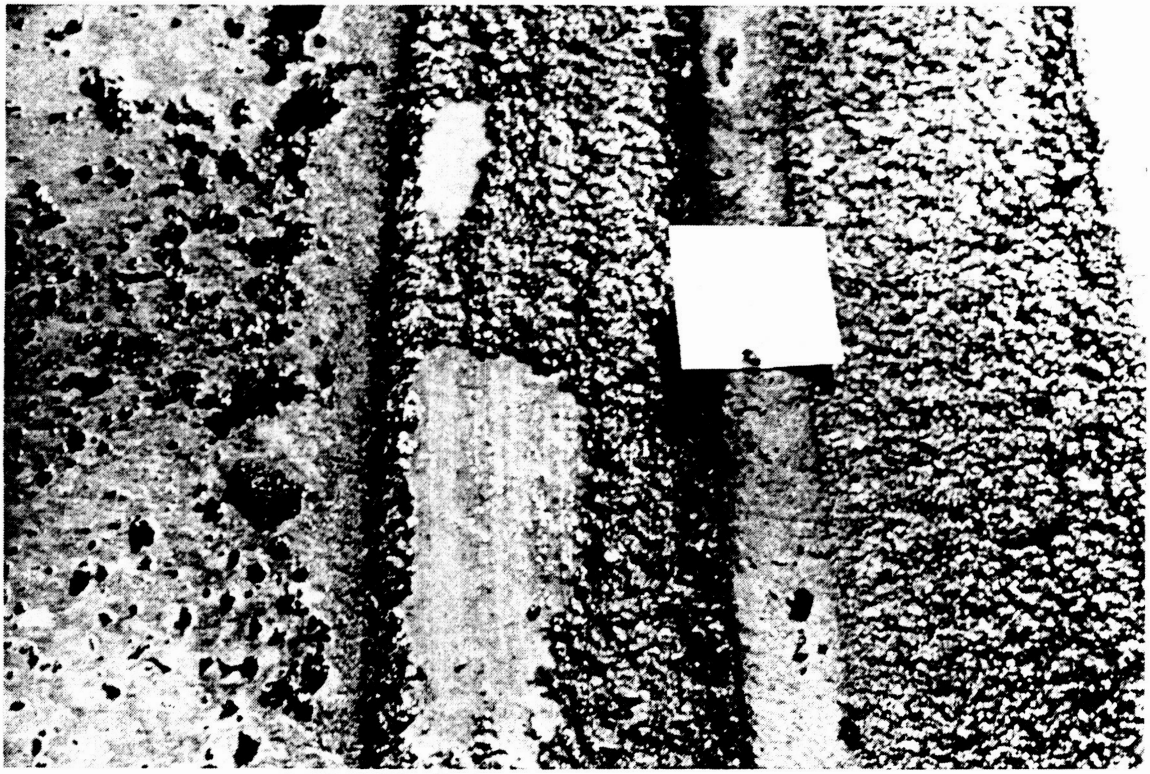
Photograph 6.19: Rubberized section R1.5 @ 2,673 reps

Photograph 6.20: Rubberized section R1.5 @ 20,864 reps



Photograph 6.21: Rubberized section R1.5 @ 28,890 reps

Photograph 6.22: Rubberized section R1.5 @ 46,135 reps

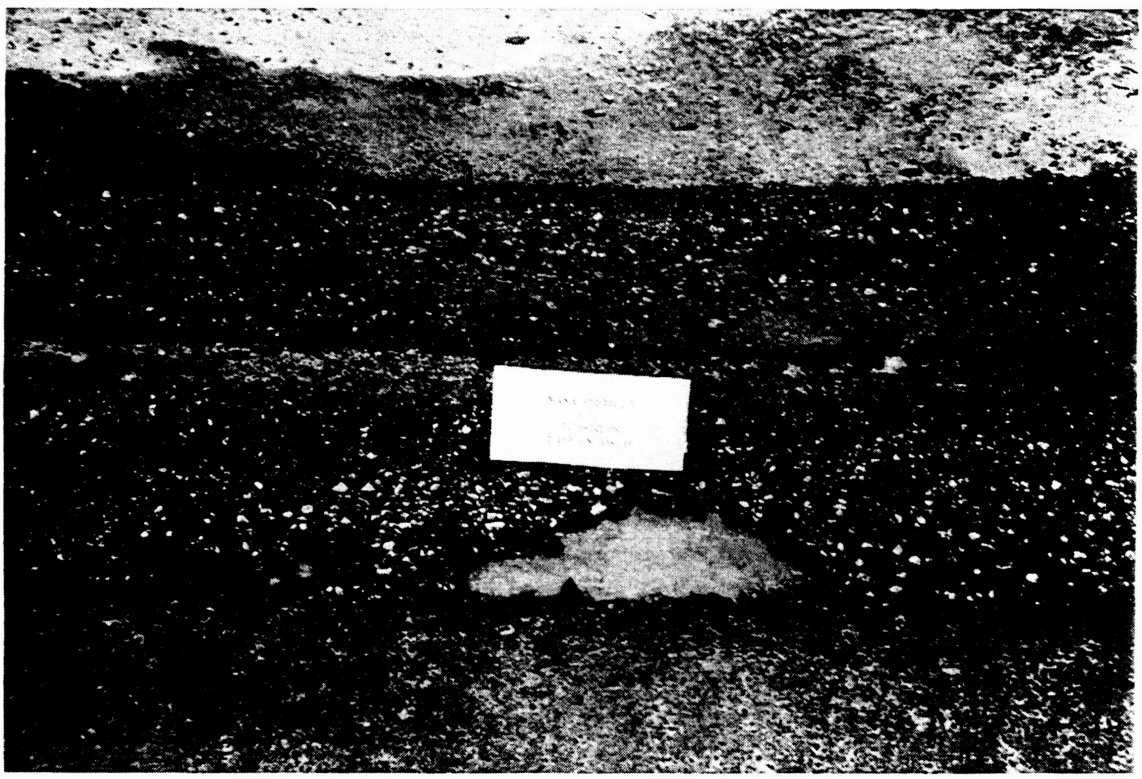


Photograph 6.23: Rubberized section R1.5 @ 55,059 reps

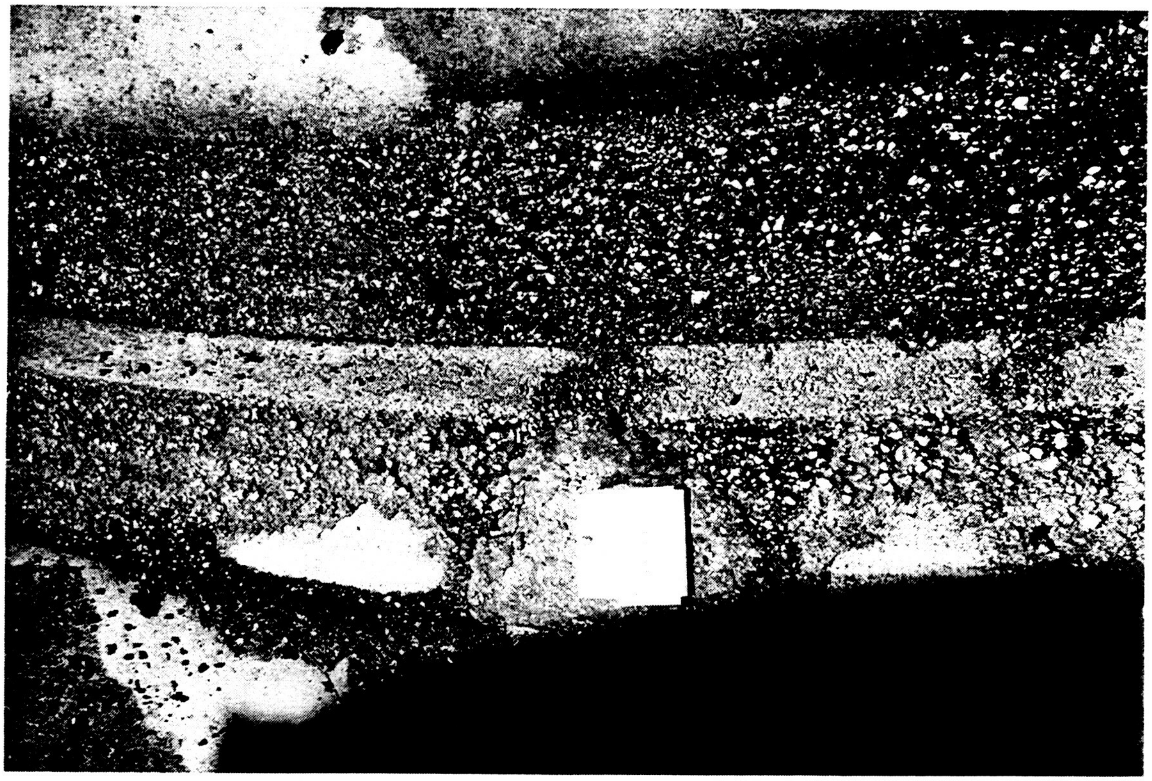
Photograph 6.24: Rubberized section R1 @ 7,684 reps

Photograph 6.25: Rubberized section R1 @ 20,864 reps

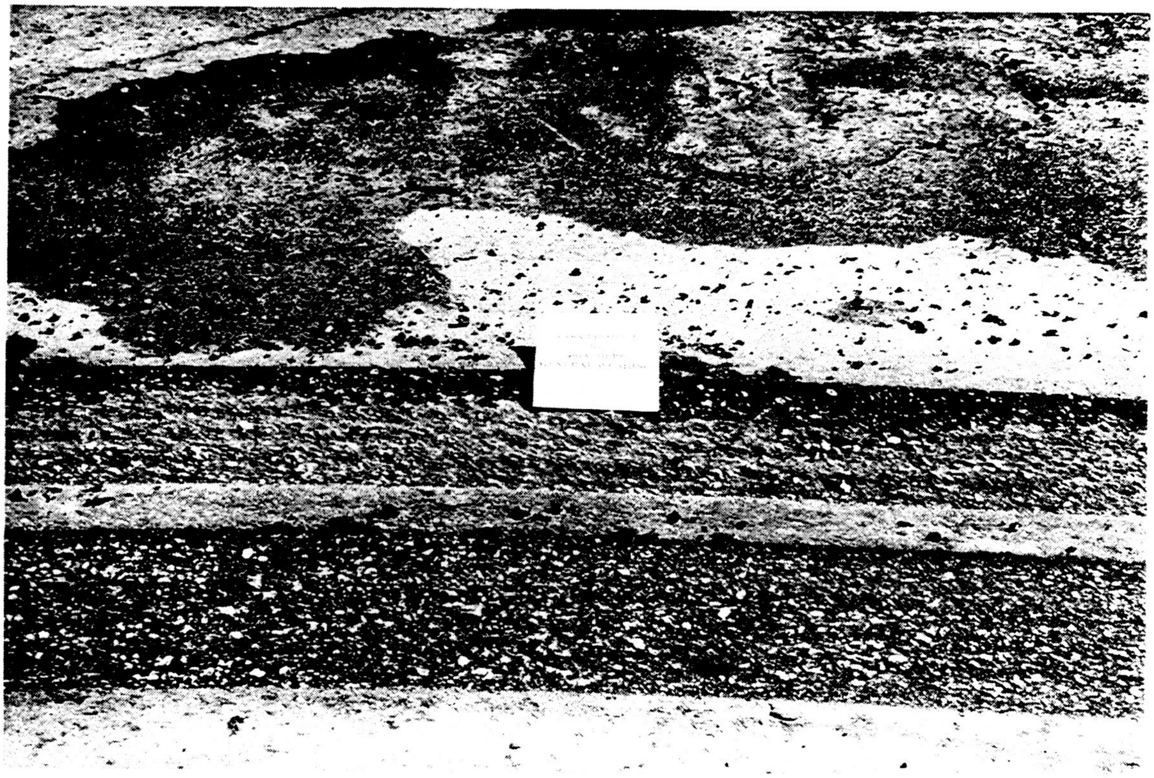
Photograph 6.26: Rubberized section R1 @ 28,890 reps



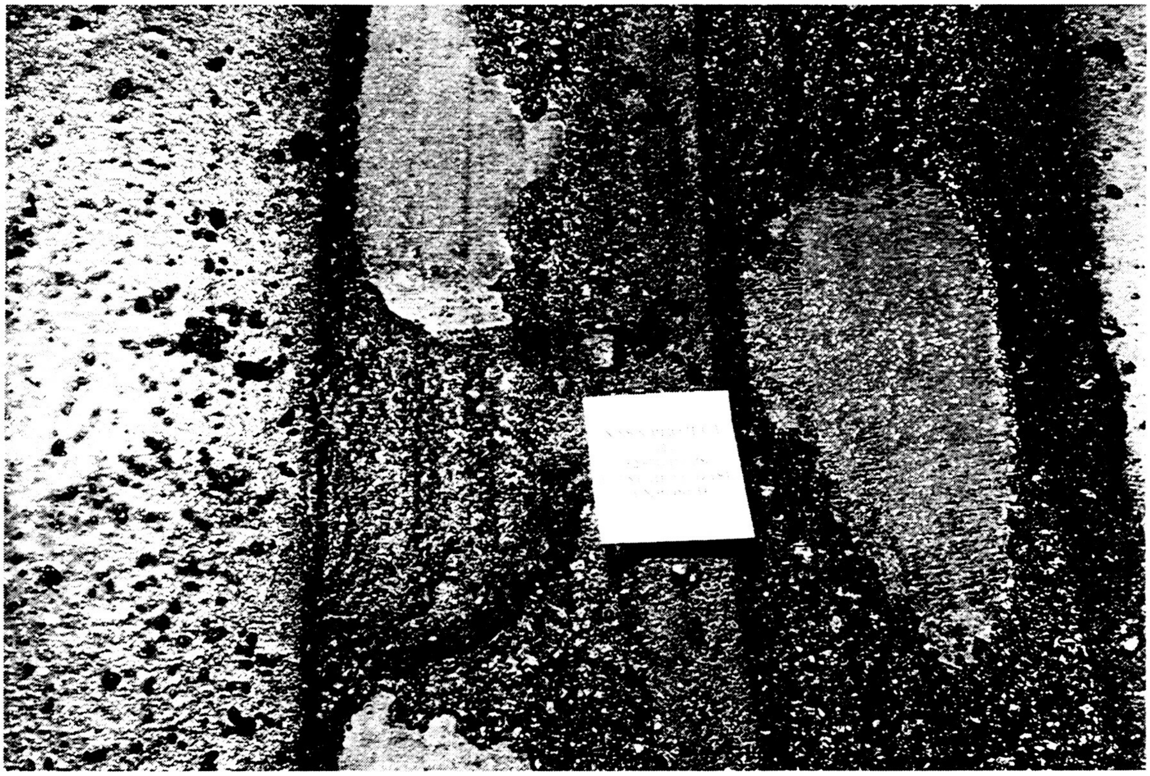
Photograph 6.27: Rubberized section R1 @ 34,306 reps (mislabeled in photo)



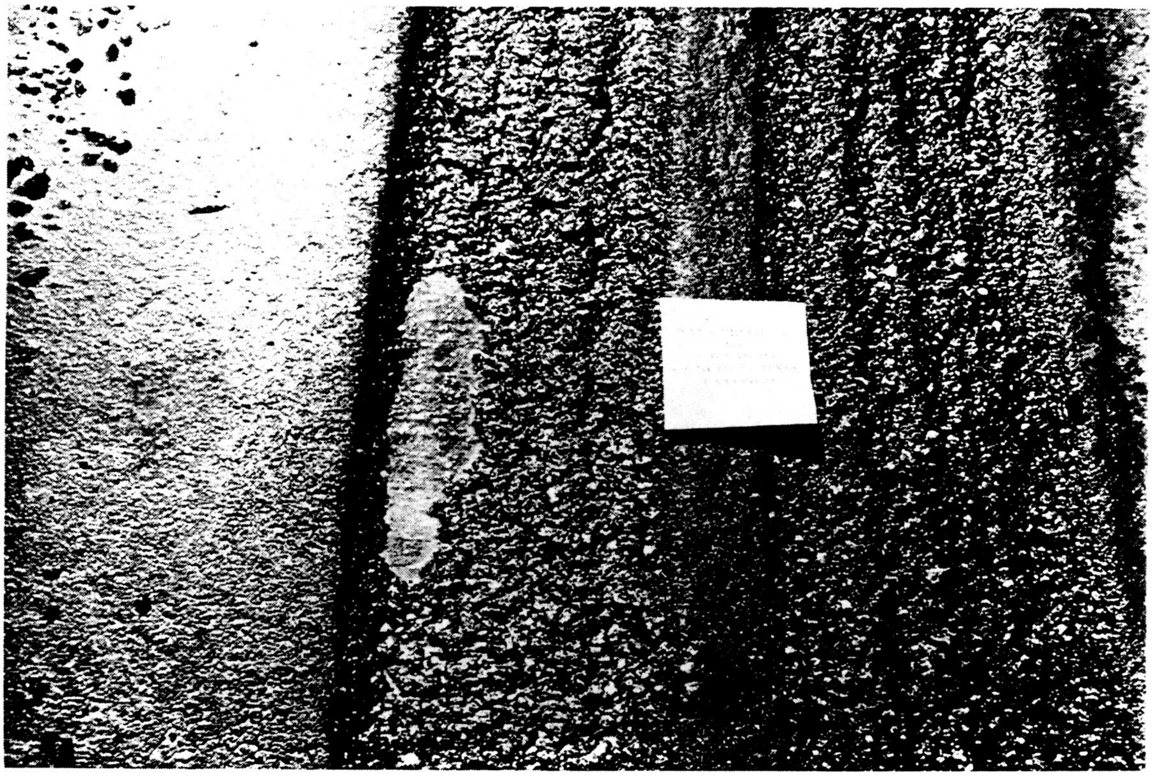
Photograph 6.28: Rubberized section R1 @ 46,135 reps



Photograph 6.29: Rubberized section R1 @ 46,135 reps



Photograph 6.30: Rubberized section R1 @ 52,578 reps



Photograph 6.31: Rubberized section R1 @ 55,059 reps

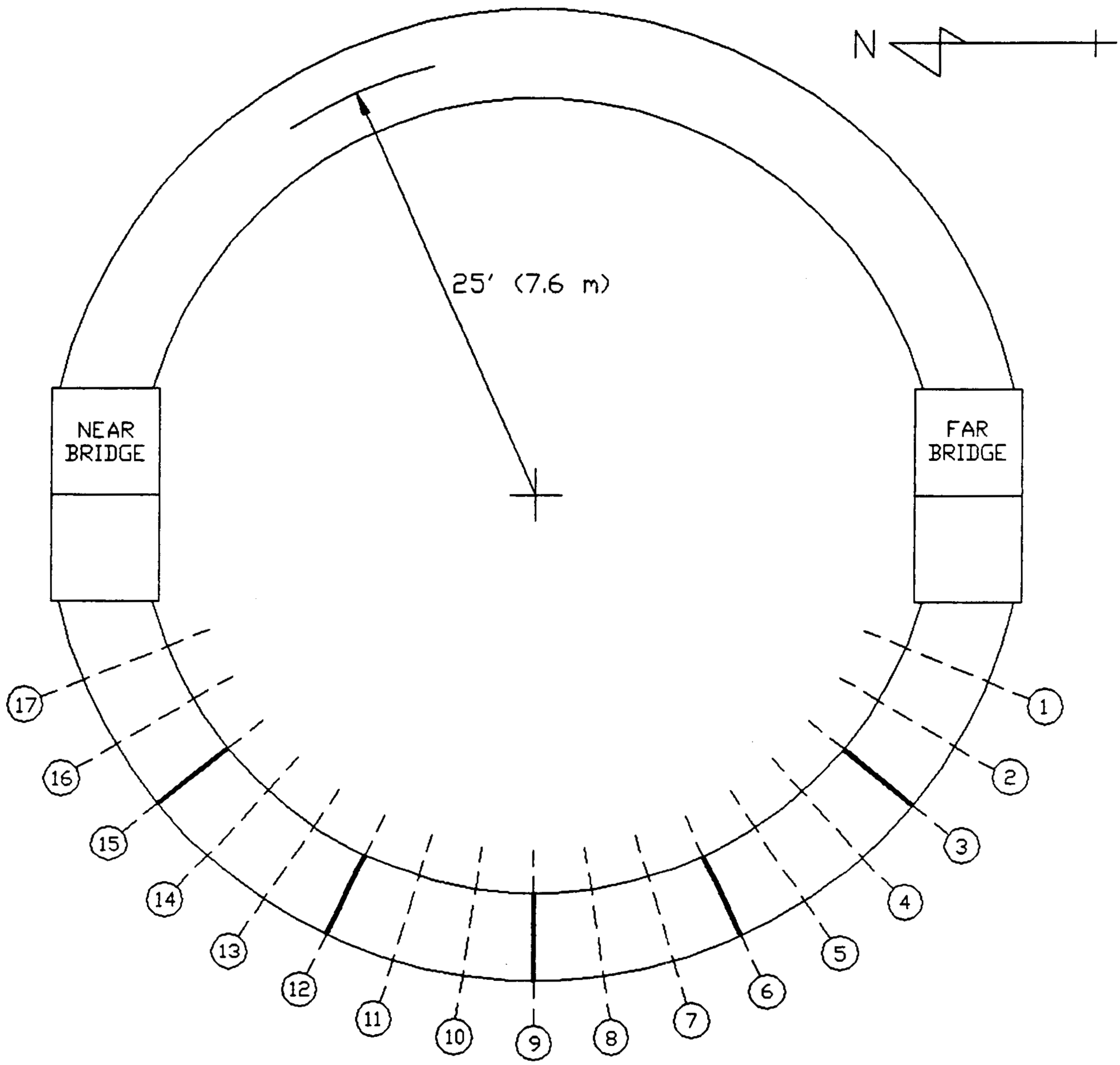


Figure 6: Locations for Rut Depth Measurements

TABLE 6: RUT DEPTHS (inches)

Location	Test Section	AT 58215 REPS Inner Wheel Path	Gap Between Wheels	Outer Wheel Path
1	R1.5	0.490	-0.203	1.479
2	R1.5	0.104	-0.172	0.396
3	R1.5-R1	0.177	-0.125	0.542
4	R1	0.896	-0.313	0.854
5	R1	0.188	-0.344	1.000
6	R1-Gripfibre	****	****	****
7	Gripfibre	0.271	-1.719	-0.177
8	Gripfibre	0.302	-0.719	0.292
9	Gripfibre-Griplast	0.313	0.203	0.344
10	Griplast	****	****	****
11	Griplast	0.313	-1.390	0.438
12	Griplast-C1	0.198	-0.422	0.271
13	C1	0.250	-0.328	0.375
14	C1	0.156	-0.344	0.313
15	C1-C1.5	0.156	-0.266	0.313
16	C1.5	0.271	-0.375	0.448
17	C1.5	0.646	-0.297	0.448

(-) = uplift
1 inch = 2.54 cm

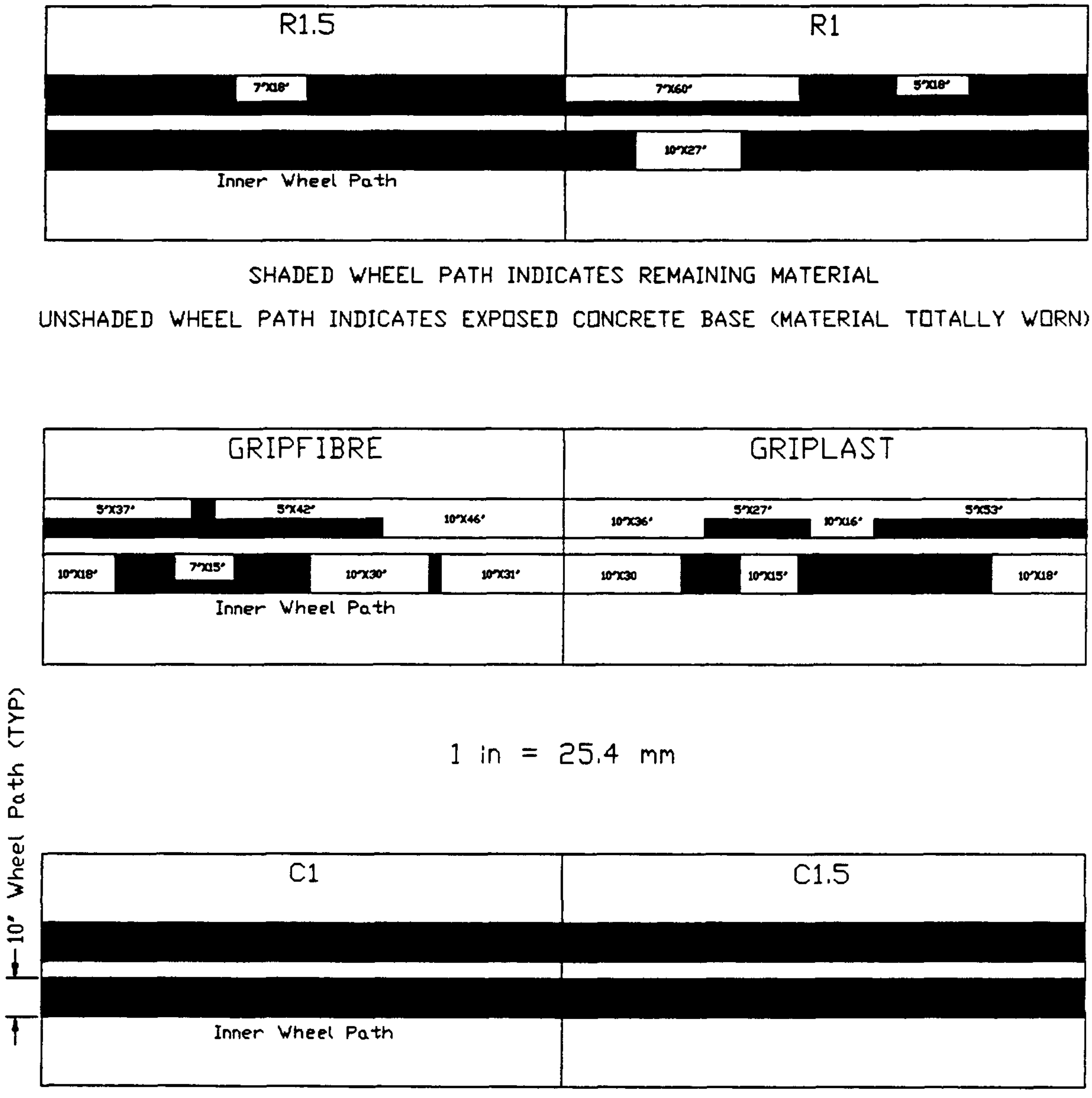


Figure 7: Pavement Distress Within the Wheel Paths (58,215 reps)

CHAPTER 7

COMPUTER PROGRAM ANALYSIS

The **CHEVRON** computer program is used for the analysis of multilayer flexible pavements. The program utilizes the elastic multilayer (five maximum) system under an equivalent circular loaded area (one load only). As output, the stresses, strains, and displacements are displayed for a number of two dimensional coordinates. The purpose of running this program is to reveal the magnitudes of stresses, strains, and displacements in the pavement components for the pavement system under study. There are other computer programs such as ELESYM 5, KENLAYER, etc. available. For simplicity, **CHEVRON 5L** was chosen.

The model used in this program consisted of four bonded layers loaded with a single uniformly circular loaded area at the top of the asphalt surface as shown in Figure 8. Even though the actual load is from a dual-wheel a single equivalent load of 11 kips (48.9 kN) is required by the program. The second layer represents the total concrete slab thickness, while the third layer represents the compacted silty sand subbase. Underneath the subbase is the natural roadbed subgrade. Coordinates of interest are at the center and edge of the load area and at the asphalt surface, bottom of asphalt, and top of the natural subgrade.

Input parameters for the Chevron program are wheel load, layer thickness, tire pressure, resilient modulus, and Poisson's ratio for each layer. By assuming proper material properties based on experience as given in Figure 8, the outputs from the analysis of the Chevron program are presented in Program Outputs. By examining the values of tangential strain (ϵ_t) at the bottom of the asphaltic layer for E_1=200 ksi (1379 MPa), which is the value needed to compute fatigue failure of the asphalt layer, it is interesting to note that the value of ϵ_t is calculated in compression rather than in tension. This means that because the concrete slab is a stiffer layer, the asphaltic layer will never suffer structural fatigue failure. Because the fatigue equations by Shell and Asphalt Institute (A.I.) rely on the tensile strain to calculate the fatigue life, they may not be applied **[15]**. The respective

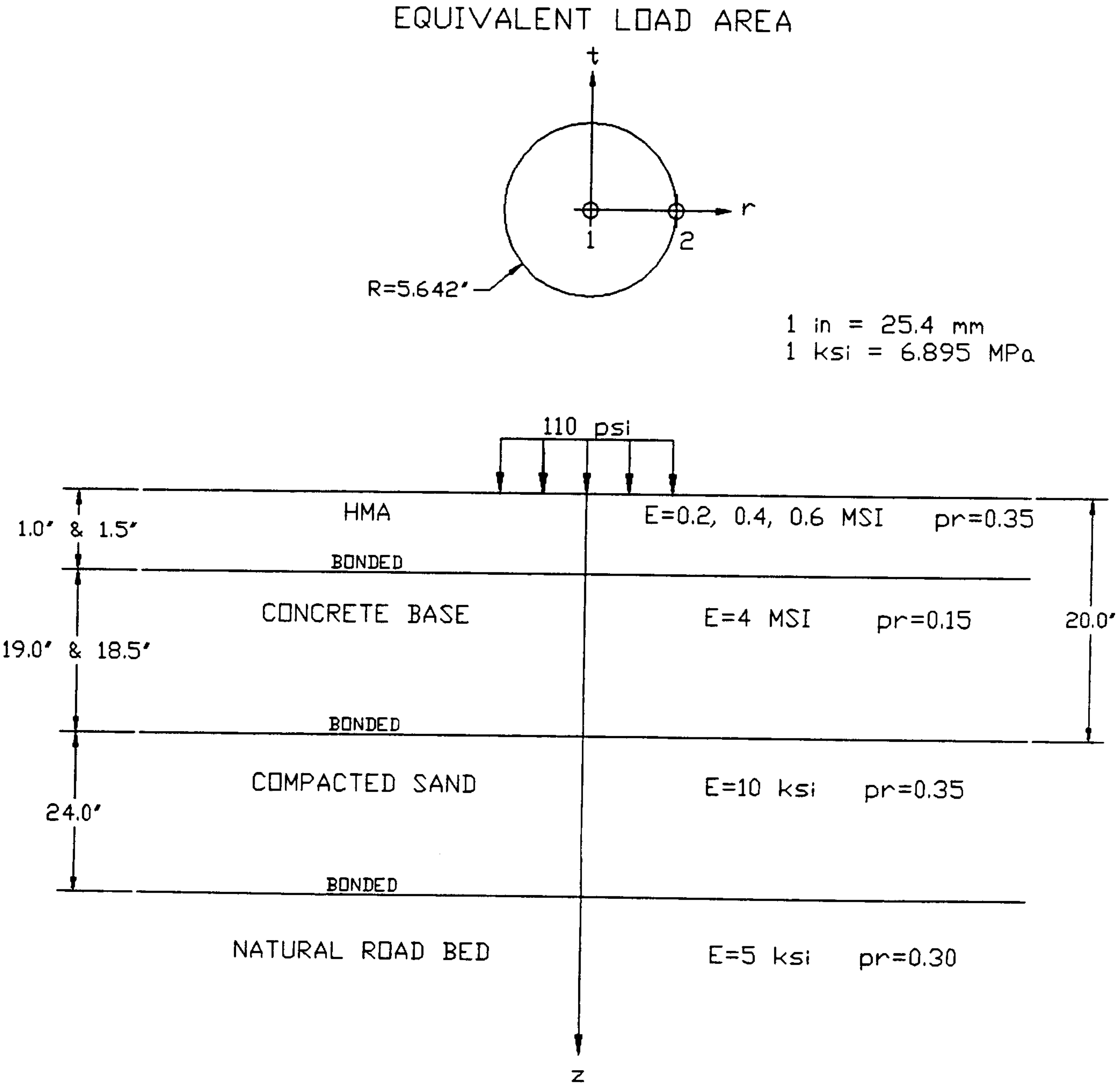


Figure 8: Chevron Computer Model

equations are: $N_f = 0.0796(\epsilon_t)^{-3.291}(E_s)^{-0.854}$ by A.I. and $N_f = 0.0685(\epsilon_t)^{-5.671}(E_s)^{-2.363}$ by Shell. Further, by examining the vertical compressive strain (ϵ_c) at the top of the subgrade from the output, the value of 2.577×10^{-5} is extremely small, which means that the pavement system will not have a functional failure of permanent deformation.

Life Expectancy Simulation Analysis

The test program was developed to continuously monitor the performance of each pavement section placed on the test track, under the application of repetitive dual wheel loading. The sum of the repetitions successfully endured is used to equate the tested paving materials to a simulated life expectancy (SLE) of normal highway use. The SLE has been tailored to site specific applications through the use of actual traffic volumes.

The first step of the simulation analysis was to determine the actual volume of yearly heavy truck traffic to which the test results were to be evaluated. Assuming an average daily traffic (ADT) volume of 7,500 for this study as in the medium-heavy traffic highway and an average percent of trucks of 6%, then the annual volume of heavy truck traffic was determined to be:

$$7{,}500 \times 0.06 \times 0.9 \times 365 \text{ days} = 147{,}825$$

where 0.9 is the multi-lane loading reduction factor as set forth by AASHTO.

For this project, a 11 kips (48.9 kN) half axle wheel load was used for the accelerated testing. This wheel loading is equivalent to a 22 kips (97.9 kN) single axle load. Therefore, it was necessary to convert this heavier wheel load to an equivalent standard wheel load of 18 kips (80.1 kN) as specified by AASHTO standard truck. An equivalent wheel load factor generally defines the damage per pass caused on a specific pavement system by the vehicle in question relative to the damage per pass of an arbitrarily selected standard vehicle moving on the same pavement system. One of the most widely used forms of load equivalency factor is that presented in the " AASHTO Guide for Design of Pavement Structures ". Based on AASHTO's convention tables from the 1986 manual, the differential equivalency factor between the 22 kip (97.9 kN) single axle load used for testing and a standard 18 kips (80.1 kN) single axle load is 2.18. The test track, unlike actual field conditions, applies the wheel loading over the exact same path during every repetition. An assumed probability of occurrence of three was used for analysis purposes. Hence, every third wheel load was assumed to cover the same path along the pavement. The following equation equates the test track results to a simulated one year life expectancy:

$$N \times 2.18 \times 3 = 147{,}825$$

where N equals the number of test track repetitions required per year. Solving for N, a total of 22,603 repetitions is obtained. The 58,215 total load repetitions run on the test track are then equivalent to approximately 2.6 years of life expectancy. However, as discussed in the previous chapter in Test Results, the pavement sections failed in much earlier time mainly due to severe ravelling and rutting from the asphalt mixtures in the surface course.

NAS

```
TOTAL LOAD.....     11000.00 LBS
TIRE PRESSURE..       110.00 PSI
LOAD RADIUS....         5.64 IN.
```

RESULTS ARE PRESENTED BELOW FOR

3 VALUES OF LAYER 1
1 VALUES OF LAYER 2
1 VALUES OF LAYER 3
1 VALUES OF LAYER 4

LAYER	MODULUS	POISSONS RATIO	THICKNESS
1	200000.	.350	1.0 IN.
2	4000000.	.150	19.0 IN.
3	10000.	.350	24.0 IN.
4	5000.	.300	SEMI-INFINITE

		S T R E S S E S					DISPLACEMENT	S T R A I N S		
R	Z	VERTICAL	TANGENTIAL	RADIAL	SHEAR	BULK	VERTICAL	RADIAL	TANGENTIAL	VERTICAL
.0	.0	-1.100E+02	-6.519E+01	-6.519E+01	.000E+00	-2.404E+02	7.565E-03	-1.938E-05	-1.938E-05	-3.218E-04
				SLOW						
.0	-1.0	-1.096E+02	-6.409E+01	-6.409E+01	.000E+00	-2.378E+02	7.242E-03	-1.655E-05	-1.655E-05	-3.235E-04
.0	1.0	-1.096E+02	-9.723E+01	-9.723E+01	.000E+00	-3.040E+02	7.242E-03	-1.655E-05	-1.655E-05	-2.010E-05
.0	-44.0	-2.706E-01	6.167E-02	6.167E-02	.000E+00	-1.472E-01	6.256E-03	1.348E-05	1.348E-05	-3.137E-05
.0	44.0	-2.706E-01	-1.968E-02	-1.968E-02	.000E+00	-3.099E-01	6.256E-03	1.348E-05	1.348E-05	-5.175E-05
5.6	.0	-7.330E+01	-4.648E+01	-5.022E+01	3.501E-05	-1.700E+02	7.314E-03	-4.147E-05	-1.626E-05	-1.973E-04

PROGRAM OUTPUT

				SLOW						
5.6	-1.0	-5.623E+01	-3.432E+01	-3.366E+01	-3.078E+01	-1.242E+02	7.125E-03	-9.845E-06	-1.430E-05	-1.622E-04
5.6	1.0	-5.623E+01	-7.448E+01	-5.899E+01	-3.078E+01	-1.897E+02	7.125E-03	-9.845E-06	-1.430E-05	-9.054E-06
5.6	-44.0	-2.693E-01	6.105E-02	6.024E-02	-8.332E-03	-1.480E-01	6.245E-03	1.331E-05	1.342E-05	-3.118E-05
5.6	44.0	-2.693E-01	-1.973E-02	-2.015E-02	-8.332E-03	-3.092E-01	6.245E-03	1.331E-05	1.342E-05	-5.147E-05

LAYER	MODULUS	POISSONS RATIO	THICKNESS
1	400000.	.350	1.0 IN.
2	4000000.	.150	19.0 IN.
3	10000.	.350	24.0 IN.
4	5000.	.300	SEMI-INFINITE

		S T R E S S E S					DISPLACEMENT	S T R A I N S		
R	Z	VERTICAL	TANGENTIAL	RADIAL	SHEAR	BULK	VERTICAL	RADIAL	TANGENTIAL	VERTICAL
.0	.0	-1.100E+02	-7.175E+01	-7.175E+01	.000E+00	-2.535E+02	7.355E-03	-2.035E-05	-2.035E-05	-1.494E-04
				SLOW						
.0	-1.0	-1.095E+02	-6.894E+01	-6.894E+01	.000E+00	-2.474E+02	7.204E-03	-1.621E-05	-1.621E-05	-1.531E-04
.0	1.0	-1.095E+02	-9.561E+01	-9.561E+01	.000E+00	-3.007E+02	7.204E-03	-1.621E-05	-1.621E-05	-2.021E-05
.0	-44.0	-2.690E-01	6.144E-02	6.144E-02	.000E+00	-1.462E-01	6.222E-03	1.341E-05	1.341E-05	-3.121E-05
.0	44.0	-2.690E-01	-1.952E-02	-1.952E-02	.000E+00	-3.081E-01	6.222E-03	1.341E-05	1.341E-05	-5.147E-05
5.6	.0	-7.330E+01	-5.183E+01	-5.412E+01	4.047E-05	-1.792E+02	7.175E-03	-2.581E-05	-1.809E-05	-9.053E-05
				SLOW						
5.6	-1.0	-5.608E+01	-3.811E+01	-3.688E+01	-3.193E+01	-1.311E+02	7.087E-03	-9.786E-06	-1.392E-05	-7.459E-05
5.6	1.0	-5.608E+01	-7.288E+01	-5.849E+01	-3.193E+01	-1.875E+02	7.087E-03	-9.786E-06	-1.392E-05	-9.094E-06
5.6	-44.0	-2.678E-01	6.082E-02	6.002E-02	-8.279E-03	-1.470E-01	6.211E-03	1.325E-05	1.336E-05	-3.101E-05
5.6	44.0	-2.678E-01	-1.957E-02	-1.998E-02	-8.279E-03	-3.074E-01	6.211E-03	1.325E-05	1.336E-05	-5.119E-05

PROGRAM OUTPUT

NAS

```
TOTAL LOAD.....    11000.00 LBS
TIRE PRESSURE..      110.00 PSI
LOAD RADIUS....        5.64 IN.
```

RESULTS ARE PRESENTED BELOW FOR

3 VALUES OF LAYER 1
1 VALUES OF LAYER 2
1 VALUES OF LAYER 3
1 VALUES OF LAYER 4

LAYER	MODULUS	POISSONS RATIO	THICKNESS
1	200000.	.350	1.5 IN.
2	4000000.	.150	18.5 IN.
3	10000.	.350	24.0 IN.
4	5000.	.300	SEMI-INFINITE

		S T R E S S E S					DISPLACEMENT	S T R A I N S		
R	Z	VERTICAL	TANGENTIAL	RADIAL	SHEAR	BULK	VERTICAL	RADIAL	TANGENTIAL	VERTICAL
.0	.0	-1.100E+02	-6.261E+01	-6.261E+01	.000E+00	-2.352E+02	8.018E-03	-1.097E-05	-1.097E-05	-3.309E-04
				SLOW						
.0	-1.5	-1.082E+02	-6.290E+01	-6.290E+01	.000E+00	-2.340E+02	7.526E-03	-1.513E-05	-1.513E-05	-3.207E-04
.0	1.5	-1.082E+02	-9.030E+01	-9.030E+01	.000E+00	-2.888E+02	7.526E-03	-1.513E-05	-1.513E-05	-2.027E-05
.0	-44.0	-2.821E-01	6.318E-02	6.318E-02	.000E+00	-1.557E-01	6.517E-03	1.398E-05	1.398E-05	-3.263E-05
.0	44.0	-2.821E-01	-2.104E-02	-2.104E-02	.000E+00	-3.242E-01	6.517E-03	1.398E-05	1.398E-05	-5.389E-05
5.6	.0	-7.330E+01	-4.657E+01	-5.085E+01	4.818E-05	-1.707E+02	7.686E-03	-4.447E-05	-1.562E-05	-1.960E-04

PROGRAM OUTPUT

				SLOW						
5.6	-1.5	-5.377E+01	-3.275E+01	-3.235E+01	-3.034E+01	-1.189E+02	7.413E-03	-1.033E-05	-1.304E-05	-1.549E-04
5.6	1.5	-5.377E+01	-6.920E+01	-5.975E+01	-3.034E+01	-1.827E+02	7.413E-03	-1.033E-05	-1.304E-05	-8.607E-06
5.6	-44.0	-2.808E-01	6.253E-02	6.169E-02	-8.709E-03	-1.566E-01	6.506E-03	1.381E-05	1.392E-05	-3.243E-05
5.6	44.0	-2.808E-01	-2.108E-02	-2.152E-02	-8.709E-03	-3.234E-01	6.506E-03	1.381E-05	1.392E-05	-5.360E-05

LAYER	MODULUS	POISSONS RATIO	THICKNESS
1	400000.	.350	1.5 IN.
2	4000000.	.150	18.5 IN.
3	10000.	.350	24.0 IN.
4	5000.	.300	SEMI-INFINITE

		S T R E S S E S					DISPLACEMENT	S T R A I N S		
R	Z	VERTICAL	TANGENTIAL	RADIAL	SHEAR	BULK	VERTICAL	RADIAL	TANGENTIAL	VERTICAL
.0	.0	-1.100E+02	-6.926E+01	-6.926E+01	.000E+00	-2.485E+02	7.695E-03	-1.630E-05	-1.630E-05	-1.538E-04
				SLOW						
.0	-1.5	-1.080E+02	-6.717E+01	-6.717E+01	.000E+00	-2.424E+02	7.463E-03	-1.463E-05	-1.463E-05	-1.525E-04
.0	1.5	-1.080E+02	-8.791E+01	-8.791E+01	.000E+00	-2.838E+02	7.463E-03	-1.463E-05	-1.463E-05	-2.041E-05
.0	-44.0	-2.796E-01	6.280E-02	6.280E-02	.000E+00	-1.540E-01	6.463E-03	1.387E-05	1.387E-05	-3.236E-05
.0	44.0	-2.796E-01	-2.077E-02	-2.077E-02	.000E+00	-3.212E-01	6.463E-03	1.387E-05	1.387E-05	-5.343E-05
5.6	.0	-7.330E+01	-5.198E+01	-5.492E+01	-3.696E-05	-1.802E+02	7.477E-03	-2.769E-05	-1.777E-05	-8.969E-05
				SLOW						
5.6	-1.5	-5.359E+01	-3.621E+01	-3.551E+01	-3.145E+01	-1.253E+02	7.351E-03	-1.020E-05	-1.256E-05	-7.122E-05
5.6	1.5	-5.359E+01	-6.711E+01	-5.891E+01	-3.145E+01	-1.796E+02	7.351E-03	-1.020E-05	-1.256E-05	-8.671E-06
5.6	-44.0	-2.784E-01	6.217E-02	6.133E-02	-8.622E-03	-1.549E-01	6.451E-03	1.370E-05	1.381E-05	-3.216E-05
5.6	44.0	-2.784E-01	-2.082E-02	-2.125E-02	-8.622E-03	-3.204E-01	6.451E-03	1.370E-05	1.381E-05	-5.315E-05

PROGRAM OUTPUT

LAYER	MODULUS	POISSONS RATIO	THICKNESS
1	600000.	.350	1.5 IN.
2	4000000.	.150	18.5 IN.
3	10000.	.350	24.0 IN.
4	5000.	.300	SEMI-INFINITE

		STRESSES					DISPLACEMENT	STRAINS		
R	Z	VERTICAL	TANGENTIAL	RADIAL	SHEAR	BULK	VERTICAL	RADIAL	TANGENTIAL	VERTICAL
.0	.0	-1.100E+02	-7.562E+01	-7.562E+01	.000E+00	-2.612E+02	7.547E-03	-1.775E-05	-1.775E-05	-9.511E-05
				SLOW						
.0	-1.5	-1.079E+02	-7.116E+01	-7.116E+01	.000E+00	-2.502E+02	7.402E-03	-1.416E-05	-1.416E-05	-9.678E-05
.0	1.5	-1.079E+02	-8.565E+01	-8.565E+01	.000E+00	-2.792E+02	7.402E-03	-1.416E-05	-1.416E-05	-2.055E-05
.0	-44.0	-2.772E-01	6.244E-02	6.244E-02	.000E+00	-1.524E-01	6.409E-03	1.376E-05	1.376E-05	-3.210E-05
.0	44.0	-2.772E-01	-2.052E-02	-2.052E-02	.000E+00	-3.183E-01	6.409E-03	1.376E-05	1.376E-05	-5.299E-05
5.6	.0	-7.330E+01	-5.718E+01	-5.886E+01	4.916E-05	-1.893E+02	7.368E-03	-2.199E-05	-1.821E-05	-5.447E-05
				SLOW						
5.6	-1.5	-5.342E+01	-3.945E+01	-3.855E+01	-3.241E+01	-1.314E+02	7.291E-03	-1.008E-05	-1.211E-05	-4.352E-05
5.6	1.5	-5.342E+01	-6.517E+01	-5.809E+01	-3.241E+01	-1.767E+02	7.291E-03	-1.008E-05	-1.211E-05	-8.732E-06
5.6	-44.0	-2.760E-01	6.181E-02	6.099E-02	-8.537E-03	-1.532E-01	6.398E-03	1.359E-05	1.371E-05	-3.190E-05
5.6	44.0	-2.760E-01	-2.056E-02	-2.099E-02	-8.537E-03	-3.175E-01	6.398E-03	1.359E-05	1.371E-05	-5.270E-05

PROGRAM OUTPUT

CHAPTER 8

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

SUMMARY

Research was conducted at the University of Central Florida by the Civil and Environmental Department involving one recycled by-product material, ground tire rubber, and one non-recycled material, microsurfacing. Following a literature search, track preparation, and material supplied by a local paving company, six pavement sections were placed on the UCF-CATT test track. Four of the pavement sections are asphalt, two conventional S-3 mixes and two rubberized mixes of the same S-3 except 5% rubber in the binder (wet process) at 1.5" and 1" thicknesses. The remaining two pavement sections are known as microsurfacing (cold slurry) and denoted as **GRIPLAST** and **GRIPFIBRE** (with fibers).

Using the UCF-CATT testing apparatus, performance testing was started on all pavement sections, running only 57.25 hours over a five week period. During this time period pavement distress was monitored and documented through photographs and rut depths measurements. All pavement sections experienced polished aggregate, rutting, material uplift, and ravelling at different times and degrees. No pavements indicated cracking except for the microsurfacing's joint reflection crack located at the track slab construction joint.

It was found that the microsurfacing was damaged in a short period of time, showing as much as 66.3% of the **GRIPFIBRE** and 58.7% of the **GRIPLAST** is worn away exposing the concrete base by the end of testing (58,215 load reps), as well as 1.7" (4.3 cm) of uplift between the dual-tires. For the **R1.5** section only 9.5% of the material is worn away to the concrete base and exhibited an average rut depth of 0.53" (1.35 cm), while its control section **C1.5** has no concrete base exposed and gave only 0.38" (0.97 cm) average rut depth. For the **R1** section 29.6% of the material is worn away to the concrete base and experienced 0.61" (1.55 cm) of average rut depth, while its control section **C1** has no concrete base exposed, giving only 0.25" (0.64 cm) average rut depth.

The **CHEVRON** computer program analysis indicated that the asphalt pavement sections will not experience horizontal tensile strain at the bottom of the asphalt layer indicating that asphalt cracking should not occur, and it did not occur during the testing period. Since the vertical compressive strain at the top of the subgrade is very small, no permanent deformation of pavement system was observed.

CONCLUSIONS

The **microsurfacing** sections failed in a short period of time. Representatives for the supplier were consulted, and they offered some of the following possible reasons for the early distresses to incur:

1. Both sections had excessive amounts of emulsion and/or water in the mixes.
2. That even though no compaction is specified in the United States, in Europe compaction is required for both the tack coating and the applied pavement to squeeze out the excess water which is blocked by the concrete slab.
3. The slow-setting SS1 tack coat should have cured for a longer period of time.
4. The water that is continuously sprayed on the track to reduce tire wear tended to pond in the ruts thereby creating water pressure in the voids and flushing the fines and larger aggregate out from the binder (ravelling).
5. Due to configuration of the test track, the paver machine's wheels had to mount the tack coated concrete surface which lifts the coating off as the wheels rotated, thus removing the bond between concrete surface and the paving material.
6. For the first half of testing, impact was induced from the dual-tires contacting the timber bridge ramps which caused the tires to bounce at certain locations around the track which corresponded to microsurfacing locations.
7. Improper or unusual installation procedures used since the materials had to handworked due to the configuration of the test track.

The asphalt pavement sections favored much better than the microsurfacing, however the control sections **C1.5** and **C1** performed better than their rubberized equivalents **R1.5** and **R1**, specifically the 1.5" (3.81 cm) thick sections. Representatives for these paving mixtures were consulted, and they also offered some of these reasons for pavement distresses incurred:

1. The water that is continuously sprayed on the track to reduce tire wear tended to pond in the ruts thereby creating water pressure in the voids (even larger for rubber mix) and flushing the fines and larger aggregate out from the binder (ravelling).
2. For the first half of testing, impact was induced from the dual-tires contacting the timber bridge ramps which caused the tires to bounce at certain locations around the track which corresponded to rubber pavement locations.
3. Due to configuration of the test track, the paver machine's wheels had to mount the tack coated concrete surface which lifts the coating off as the wheels rotated, thus removing the bond between concrete surface and the paving material.

RECOMMENDATIONS

From the results of this study, the following recommendations are given:

1. Rubberized asphalt (5% rubber by wt. of binder) should be at least 1.5" (3.81 cm) thick for all pavement system.
2. More research is needed regarding the material testing and installation procedures for the microsurfacing products to be overlayed on concrete pavements.
3. Further, more testing is required for a number of material sets both from laboratory specimens and core sampling in order to correlate the performance with optimum content of additives in paving mixture, specifications, and cost effectiveness.

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LAYER	MODULUS	POISSONS RATIO	THICKNESS
1	600000.	.350	1.0 IN.
2	4000000.	.150	19.0 IN.
3	10000.	.350	24.0 IN.
4	5000.	.300	SEMI-INFINITE

		STRESSES					DISPLACEMENT	STRAINS		
R	Z	VERTICAL	TANGENTIAL	RADIAL	SHEAR	BULK	VERTICAL	RADIAL	TANGENTIAL	VERTICAL
.0	.0	-1.100E+02	-7.810E+01	-7.810E+01	.000E+00	-2.662E+02	7.260E-03	-2.044E-05	-2.044E-05	-9.222E-05
				SLOW						
.0	-1.0	-1.095E+02	-7.360E+01	-7.360E+01	.000E+00	-2.566E+02	7.165E-03	-1.588E-05	-1.588E-05	-9.656E-05
.0	1.0	-1.095E+02	-9.405E+01	-9.405E+01	.000E+00	-2.975E+02	7.165E-03	-1.588E-05	-1.588E-05	-2.031E-05
.0	-44.0	-2.676E-01	6.121E-02	6.121E-02	.000E+00	-1.451E-01	6.189E-03	1.334E-05	1.334E-05	-3.104E-05
.0	44.0	-2.676E-01	-1.936E-02	-1.936E-02	.000E+00	-3.063E-01	6.189E-03	1.334E-05	1.334E-05	-5.119E-05
5.6	.0	-7.330E+01	-5.702E+01	-5.793E+01	4.795E-05	-1.882E+02	7.103E-03	-2.053E-05	-1.849E-05	-5.511E-05
				SLOW						
5.6	-1.0	-5.594E+01	-4.173E+01	-4.002E+01	-3.295E+01	-1.377E+02	7.050E-03	-9.723E-06	-1.357E-05	-4.555E-05
5.6	1.0	-5.594E+01	-7.138E+01	-5.799E+01	-3.295E+01	-1.853E+02	7.050E-03	-9.723E-06	-1.357E-05	-9.134E-06
5.6	-44.0	-2.664E-01	6.060E-02	5.981E-02	-8.228E-03	-1.459E-01	6.178E-03	1.318E-05	1.329E-05	-3.085E-05
5.6	44.0	-2.664E-01	-1.940E-02	-1.982E-02	-8.228E-03	-3.056E-01	6.178E-03	1.318E-05	1.329E-05	-5.092E-05

PROGRAM OUTPUT